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INTEGRATED-BIOLOGICAL-CHEMICAL WASTEWATER TREATMENT

C. F. Garland, et al

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INTEGRATED BIOLOGICAL-CHEMICAL WASTEWATER TREATMENT

by

C. F. Garland and G. L. Shell

for

Advanced Waste Treatment Research Activities
Federal Water Pollution Control Administration
United States Department of the Interior

November, 1966

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ABSTRACT

Conceptual development of an integrated biological-chemical treatment system for organic wastewaters is described. A system of this type comprising a highly-loaded, short-detention activated sludge system followed by chemical coagulation and clarification is evaluated in comparison with the traditional activated sludge process.

The following principal conclusions are developed from a series of bench- and pilot-scale studies of treatment of a weak domestic sewage:

1. First-stage effluent quality is inversely related to load intensity.
2. First stage process stability is unaffected at high loads.
3. Second-stage coagulant demand to achieve a given effluent quality is a direct function of first-stage loading.
4. For the substrate studied, appreciable coagulant dosages are needed to obtain final effluent quality comparable to that from the traditional activated-sludge controls.
5. Relatively large quantities of waste sludge are generated in such a treatment system. This aspect of specific potential applications of the process will require careful definition.
6. In comparison with traditional activated-sludge treatment, the process offers important advantages in stability and flexibility and is capable of producing as good or better effluent quality.

A cost analysis discloses that the process is competitive.

INTRODUCTION

Water requirements for future populations will be met, to an ever-increasing degree, by reuse of wastewaters. Presently, a significant fraction of wastewaters of varying origin may constitute a portion of river water supply to downstream users. Normal waste treatment processes, supplemented by dilution and natural purification, are being increasingly taxed in their ability to produce an unobjectionable water supply at downstream points. Improvements in wastewater treatment methods are required to avoid even more objectionable conditions in the future.

Ultimately, water demands may be such that wastewater renovation and reuse become both practicable and economical for municipal systems. Discharge from such systems will comprise the consumptive use, losses, and that contained in renovation-process waste concentrates. Currently, treated municipal wastewaters are being used for industrial purposes at several locations. Special additional treatment is required in most cases preceding this reuse. The degree of further treatment required is, of course, dictated by the use to be made of the renovated wastewater.

Improvement of present wastewater treatment efficiency, even though short of producing water of suitable final quality, could be integrated with additional treatment for the most economical processing for direct reuse. Accordingly, this research was undertaken to evaluate a treatment process offering such potential.

Background

The activated sludge process is the most efficient treatment system now in general use for removal of dissolved organic matter. Treatment is visualized as occurring in two principal phases: (1) substrate utilization, and (2) bioflocculation. The organic content of the waste is first utilized as food by the microorganisms responsible for treatment, a portion of the removed substrate being synthesized to new cell material while the remainder is oxidized to provide energy for synthesis. Rapid substrate utilization is promoted by a high food-to-microorganism (F/M) ratio, an environment in which the newly synthesized microbial cells are largely dispersed. Removal of these cells is required for maximum over-all treatment plant performance, otherwise treatment is limited to the respired fraction of the original organic content.

This is accomplished in an exclusively biological system by providing an environment which fosters flocculation, the second treatment phase. Bioflocculation is not well understood, but the influence of such factors as the growth stage and energy content of the activated sludge is recognized and flocculation is observed to occur best under conditions favoring endogenous metabolism.

In practice, treatment is effected in large and costly aeration basins in which the conflicting requirements for full development of the potential of the two process phases are compromised. Process improvement therefore entails consideration of how to enhance the efficiency of the two treatment mechanisms.

The work of many investigators has shown that under proper conditions substrate utilization is very rapid. Good biological removal of soluble organics can be realized in much less time (smaller aeration basins) than is presently the case in practice if bioflocculation is ignored. It should then be possible to chemically flocculate the suspended content of effluent from such a system, including the dispersed microbial population generated by first-stage operation at a high F/M ratio, at reasonable cost. Efficient removal of suspended and colloidal materials from domestic and industrial wastewaters by chemical flocculation and sedimentation has long been practiced and is finding current application as a method of further improving the quality of effluent from conventional activated-sludge and trickling-filter treatment systems.

In addition, an integrated treatment system of this type could be operated with the supplemental and increasingly important objective of phosphate reduction.

Hopefully, the reduced cost of biological treatment would largely balance the cost of chemical treatment, enabling production of treated wastewater of equal or better quality than that from exclusively biological treatment at essentially the same cost.

Process Description

In concept, the process is two-stage in nature. The initial stage is a highly-loaded, short-detention activated sludge system, including settling and sludge recirculation. It may or may not be preceded by primary treatment and is operated at loadings which do not necessarily produce floc with optimum settling characteristics.

The second stage comprises coagulation and clarification using chemicals and biological suspended matter (activated sludge) for floc production, the latter probably in the form of the suspended solids content of the first-stage effluent. Excess sludge from the first stage may be wasted to the second stage.

Experimental Approach

Process evaluation was divided into two parts: (I) a series of bench studies designed to disclose the relationship of activated sludge loadings and the corresponding required chemical treatment to optimize final effluent quality; and (II) operation of a pilot plant to confirm on a larger scale the most promising results from the laboratory investigations.

Initial consideration was given the idea of using a reproducible synthetic substrate for at least the bench-scale program to eliminate the inevitable complications of working with feed of variable strength. But treatment of an actual wastewater seemed more advantageous.

Because treatment of domestic sewage would represent an important application of the process and because settled sewage was readily available, this type of wastewater was selected for study.

The pilot plant was located at the Municipal Sewage Treatment Works serving the City of Tucson, Arizona, where it was presumed the traditionally designed 12-mgd activated sludge portion of the plant would serve as a control.

Inasmuch as considerable variation in feed strength and results could be expected, it was appreciated that a significant number of samples and a marked difference in their distribution were required for reaching statistically sound conclusions as to treatment differences. In view of the number of variables involved, however, some compromise from an ideal program was necessary, and it was decided to accumulate data over 10 to 15 days for each experiment.

Analytical Methods

With certain exceptions, methods used in this study conformed to those in "Standard Methods."[3] Total Organic Carbon was determined at the Taft Sanitary Engineering Center using a Beckman Carbonaceous Analyzer. Filtered samples were acidified to pH 2.0 prior to shipment.

The mercuric sulfate modification of the COD procedure and the brucine method for nitrate were used. Suspended matter was determined by filtration through Whatman GF/A glass paper in a Gooch crucible. The methyl green method was used to evaluate surfactant (ABS) concentrations.

EXPERIMENTAL INVESTIGATIONS

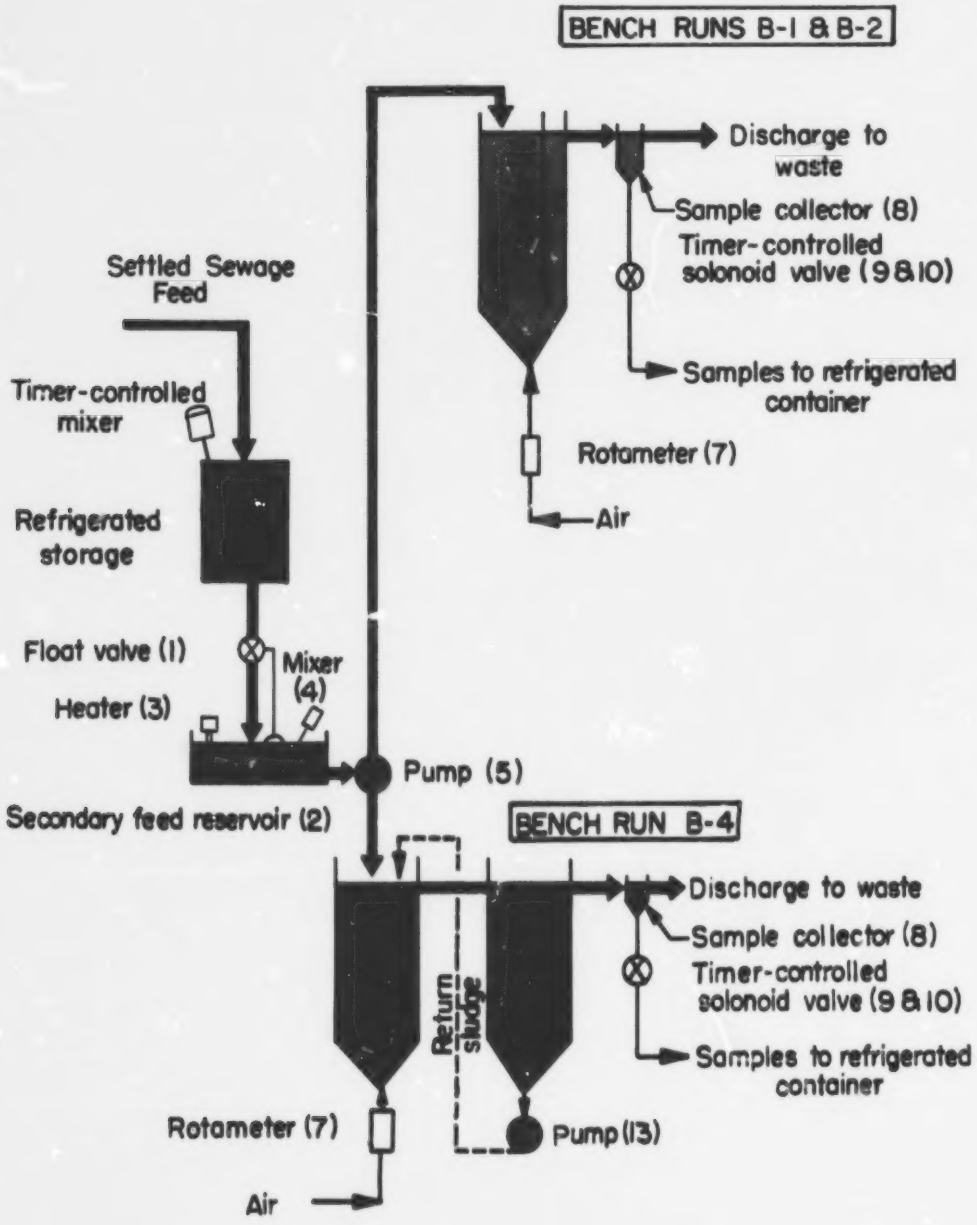
PART I: BENCH STUDIES

Apparatus and Procedure

Settled sewage, collected once each day during the peak strength period at the sewage treatment plant, was stored below 5°C in a 200-gal refrigerated feed storage tank which was drained and rinsed prior to each recharging. Tank contents were thoroughly mixed for 1 minute at 15-minute intervals with a timer-controlled propeller mixer to keep solids in suspension.

The bench systems are shown in Figures 1 and 2. Waste from the primary storage tank discharged by gravity through a float valve (1) into a secondary feed reservoir (2) in which its temperature was raised to 25°C by thermostatically-controlled heaters (3) and in which uniform conditions were maintained by a small air-operated propeller mixer (4).

Peristaltic feed pumps (5) transferred waste at selected constant rates from the secondary storage tank to as many as seven 5.5-liter completely-mixed activated sludge units (6) of the type described by Ludzack[1/]. Individual composite 24-hr effluent samples were collected from these by a special sampling device comprising a small plastic collector (8) and solenoid valve (10) actuated by a timer (9). Approximately 65 ml of sample were drained from the collector at 5-min intervals to a refrigerated 5-gal plastic container. Excess effluent overflowed to waste from the collector.



SCHEMATIC : ACTIVATED SLUDGE BENCH APPARATUS

Figure I

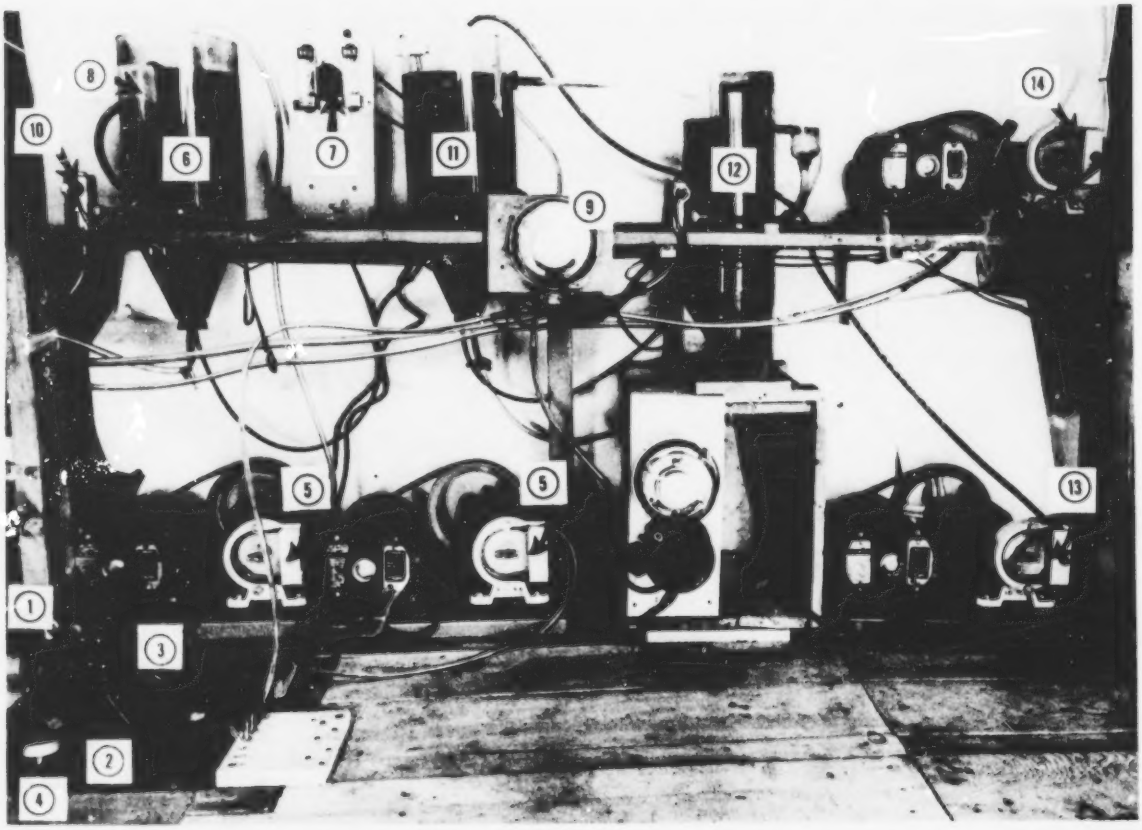


Figure 2.

LABORATORY ACTIVATED SLUDGE AND SAMPLING SYSTEMS.

Air supplied at 1.0 scfh through a rotameter (7) and porous stone diffuser provided adequate oxygenation and mixing in each unit at all but the highest loading studied.

Mixed-liquor suspended solids (MLSS) were controlled by wasting sludge once daily. Changes in sludge characteristics were followed daily by MLSS and Sludge Volume Index (SVI) determinations plus microscopic observation.

The primary feed storage tank was sampled twice daily for analysis, once after charging and again just before draining. Feed strength was taken as the average of results so obtained.

Because of adverse hydraulic conditions associated with use of the combined aeration-settling units at the highest loading studied, different apparatus was required. An aeration vessel (11) overflowed by gravity to a separate clarifier (12) from which sludge was recycled to the aeration unit by a timer-controlled pump (13). A similar pumping arrangement was used to control sludge wasting. Additional air and diffusers were required to maintain dissolved oxygen in the aeration vessel. During this run a third timer-controlled pump (14) collected a composite 24-hr feed sample from the secondary feed reservoir.

It is well established that the liquid detention time in the aeration basin component of an activated sludge system is not fundamental and can be as low as 30 minutes in the treatment of normal domestic wastewater. Most important is the F/M weight ratio, commonly expressed in terms of lb BOD/lb MLSS/day. In this research, selected first-stage

load ratios were obtained by adjustment of feed flow rates and MLSS concentrations, with due regard, however, for the minimum liquid detention period. The units were operated in parallel to develop comparative data. A control unit functioned at the traditional loading of about 0.4 lb BOD/lb MLSS/day and a 6-hr aeration time (based on feed rate).

Analytical data accumulated daily during each test run of several days' duration normally included COD, total organic carbon (TOC), BOD, suspended matter, pH, and temperature. On the basis that effluent quality would be best judged by the soluble organics present, COD and TOC were commonly determined after filtering the samples through Whatman GF/A glass paper. Non-routine determinations included ABS, total phosphate, and nitrogen (organic, ammonia, and nitrate).

Chemical flocculation of the composited effluent from each activated sludge unit was evaluated by jar testing, using the gang stirrer and special sampling apparatus described by Cohen[2/] Chemicals were dosed as desired to six 1750-ml samples in individual 2000-ml beakers, flash mixed one minute at 100 rpm, flocculated nine minutes at 30 rpm, and settled 30 minutes. During early phases of the investigation the beakers were sampled simultaneously at intervals during settling. Later, it was deemed sufficient to determine supernatant quality at the end of the settling period.

Much of the flocculation testing was exploratory in nature and extensive supernatant analysis was not required. However, supernatant from the most promising tests received more complete analysis.

Experimental Results

Activated Sludge Treatment

Three exploratory runs were conducted, each preceded by at least two weeks of acclimation at the F/M loadings selected for evaluation.

Run B-1 totaled 17 days, on 12 of which samples were collected for analysis. Run B-2, designed to cover a wider F/M range, totaled 20 days, on 15 of which samples were obtained. During Run B-4 in which only two activated sludge systems were operated, elapsed time was 24 days, including 18 sample days.

Pertinent information for each run is summarized in Tables 1, 2, 3, and Figure 3. It is evident that the strength of feed sewage differed significantly among the runs, invalidating direct comparison of effluent quality. However, the influence of system loading on activated-sludge effluent characteristics is very apparent in Figure 3 and it was obvious that effluent degradation could be expected at any loading much above that of the conventional process. But since it is known that chemical treatment of such effluents will effect further removals, there was reason to feel that equivalent final quality could be achieved with the combination process and this was verified as the program developed.

Plant-scale activated sludge treatment of normal strength domestic sewage generally accomplishes about 85 per cent BOD removal across the aeration basin and final clarifier.* It will be noted

---

*The reader's attention is invited to the use of data on primary effluent for removal efficiency computations throughout this report, rather than overall removal from raw to final effluent.

TABLE 1

Average Performance Data. Activated-Sludge Bench Units. Run B-1

Item	Unit Number 1	2	3	4	5	6	7
Load, mg BOD/mg MLSS/day	0.4	0.5	0.6	0.7	1.2	1.5	1.4
Feed rate, ml/min	13	25	25	25	51	51	51
Aeration time, min	360	186	186	186	91	91	91
Air rate, SCFH	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Clarification							
detention time, min	62	32	32	32	16	16	16
overflow rate, gpd/ft^2	180	340	340	340	700	700	700
Temperature, ^oC	24	24	24	24	24	24	24
pH	7.6	7.9	7.9	7.9	7.9	7.8	7.8
MLSS, mg/l	1140	1530	1300	1120	1430	1080	1150
Sludge Volume Index, ml/l	71	314	398	360	168	99	153
Biochemical Oxygen Demand, mg/l							
influent	124	124	124	124	124	124	124
effluent	21	20	34	19	36	45	36
% removal	84	83	72	84	71	64	71
mg fed/day	2320	4460	4460	4460	9110	9110	9110
mg removed/day/mg MLSS	0.31	0.44	0.45	0.62	0.82	0.98	1.02
Chemical Oxygen Demand, mg/l							
influent, unfiltered	264	264	264	264	264	264	264
effluent, filtered	54	53	48	48	77	83	76
Total Organic Carbon, mg/l							
influent, filtered	65	65	65	65	65	65	65
effluent, filtered	14	15	13	13	20	24	20
% removal	78	77	80	80	69	63	69
Suspended Solids, mg/l							
influent	105	105	105	105	105	105	105
effluent	22	35	35	23	34	45	35
% removal	79	67	67	73	68	57	67

TABLE 2

Average Performance Data. Activated-Sludge Bench Units. Run B-2

Item	Unit Number 1	2	3	4	5	6	7
Load, mg BOD/mg MLSS/day	0.4	1.1	1.5	2.6	2.9	3.4	3.4
Feed rate, ml/min	13	30	30	55	55	55	55
Aeration time, min	360	144	144	84	84	84	84
Air rate, SCFH	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Clarification							
detention time, min	62	27	27	15	15	15	15
overflow rate, gpd/ft^2	180	410	410	750	750	750	750
Temperature, ^{o}C	22	22	22	22	22	22	22
pH	7.4	7.9	7.9	7.8	7.8	7.8	7.8
MLSS, mg/l	1350	1110	850	900	780	670	670
Sludge Volume Index, ml/l	177	248	113	95	88	58	88
Biochemical Oxygen Demand, mg/l							
influent	158	158	158	158	158	158	158
effluent	38	36	40	64	57	58	48
% removal	76	77	75	59	64	63	70
mg fed/day	2960	6840	6840	12500	12500	12500	12500
mg removed/day/mg MLSS	0.30	0.86	1.09	1.51	1.86	2.15	2.36
Chemical Oxygen Demand, mg/l							
influent, filtered	154	154	154	154	154	154	154
effluent, filtered	39	61	75	91	97	97	89
% removal	75	60	51	41	37	37	42
Total Organic Carbon, mg/l							
influent, unfiltered	89	89	89	89	89	89	89
effluent, filtered	14	20	24	28	28	30	28
Suspended Solids, mg/l							
influent	102	102	102	102	102	102	102
effluent	32	34	40	56	58	56	52
% removal	69	67	61	45	43	45	49

TABLE 3

Average Performance Data
Activated-Sludge Bench Units. Run B-4

Item	Unit Number 1	2
Load, mg BOD/mg MLSS/day	0.4	4.8
Feed rate, ml/min	14	120
Aeration time, min	336	39
Air rate, SCFH	1.5	6.0
Clarification		
detention time, min	58	46
overflow rate, gpd/ft^2	190	230
Temperature, ^oC	23	23
pH	7.6	7.7
MLSS, mg/l	960	760
Sludge Volume Index, ml/l	173	57
Biochemical Oxygen Demand, mg/l		
influent	113	113
effluent	26	48
% removal	77	58
mg fed/day	2280	19500
mg removed/day/mg MLSS	0.30	2.7
Chemical Oxygen Demand, mg/l		
influent, filtered	143	143
effluent, filtered	49	94
% removal	66	34
Total Organic Carbon, mg/l		
influent, filtered	46	46
effluent, filtered	21	29
% removal	54	37
Suspended Solids, mg/l		
influent	73	73
effluent	31	47
% removal	58	35
Total Phosphate, mg/l as PO_4^{3-}		
influent	42	42
effluent	39	40

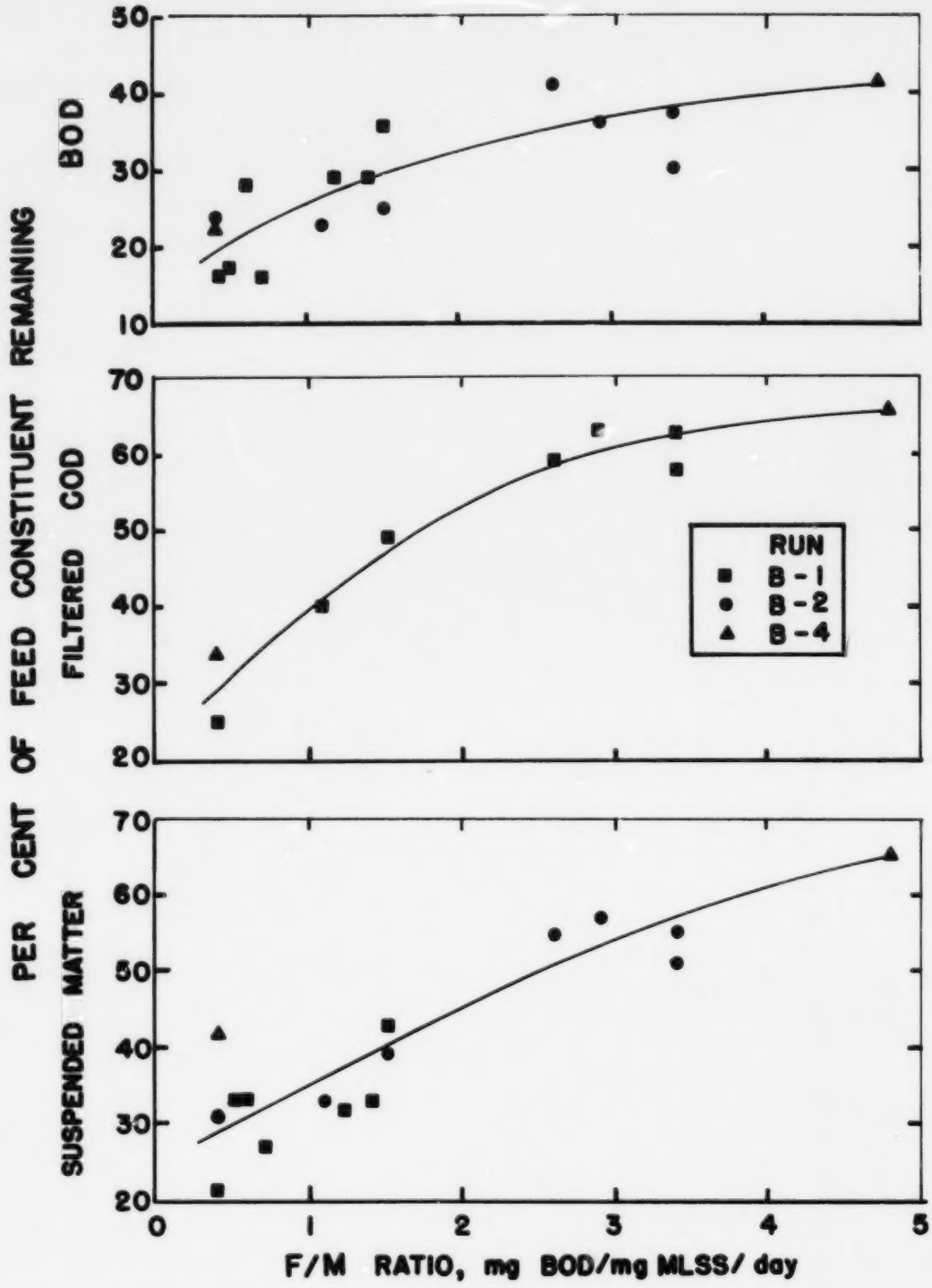


Figure 3

ACTIVATED SLUDGE EFFLUENT QUALITY
BENCH STUDY

that somewhat less efficiency characterized performance of the laboratory units, probably reflecting scale and operational effects as much as anything else.

The improved effluent quality observed at an F/M ratio of 3.4 during Run B-2 is illogical and difficult to explain. Aeration time and hydraulic loading in the clarification section of Units 6 and 7 were no different than in Units 4 and 5 which were loaded nearly as heavily. However, the settleability of sludge in Unit 6 was much better and this probably accounts for the improved performance of this unit.

The same is true for Unit 2 of Run B-4 loaded at an F/M ratio of 4.8. The lower feed strength, lower clarifier overflow rate, and closer operational control of different experimental apparatus were additional factors which influenced the improved effluent quality developed during this run.

The cumulative curves of Figure 4 show effluent quality and its dispersion at F/M ratios of 2.6 and 4.8, together with similar information for the respective controls. While it might be expected that system sensitivity would increase with load intensity, it is clear that process stability was virtually unaffected at high loads. Indeed, performance of the highly loaded systems was less variable than that of the conventionally loaded controls.

It is of interest to note in Figure 5 the enhancement of sludge activity with increased load. At an F/M ratio of 4.8, the maximum load studied, nearly 10 times as much BOD was removed in a given period per unit weight of sludge as at conventional loading.

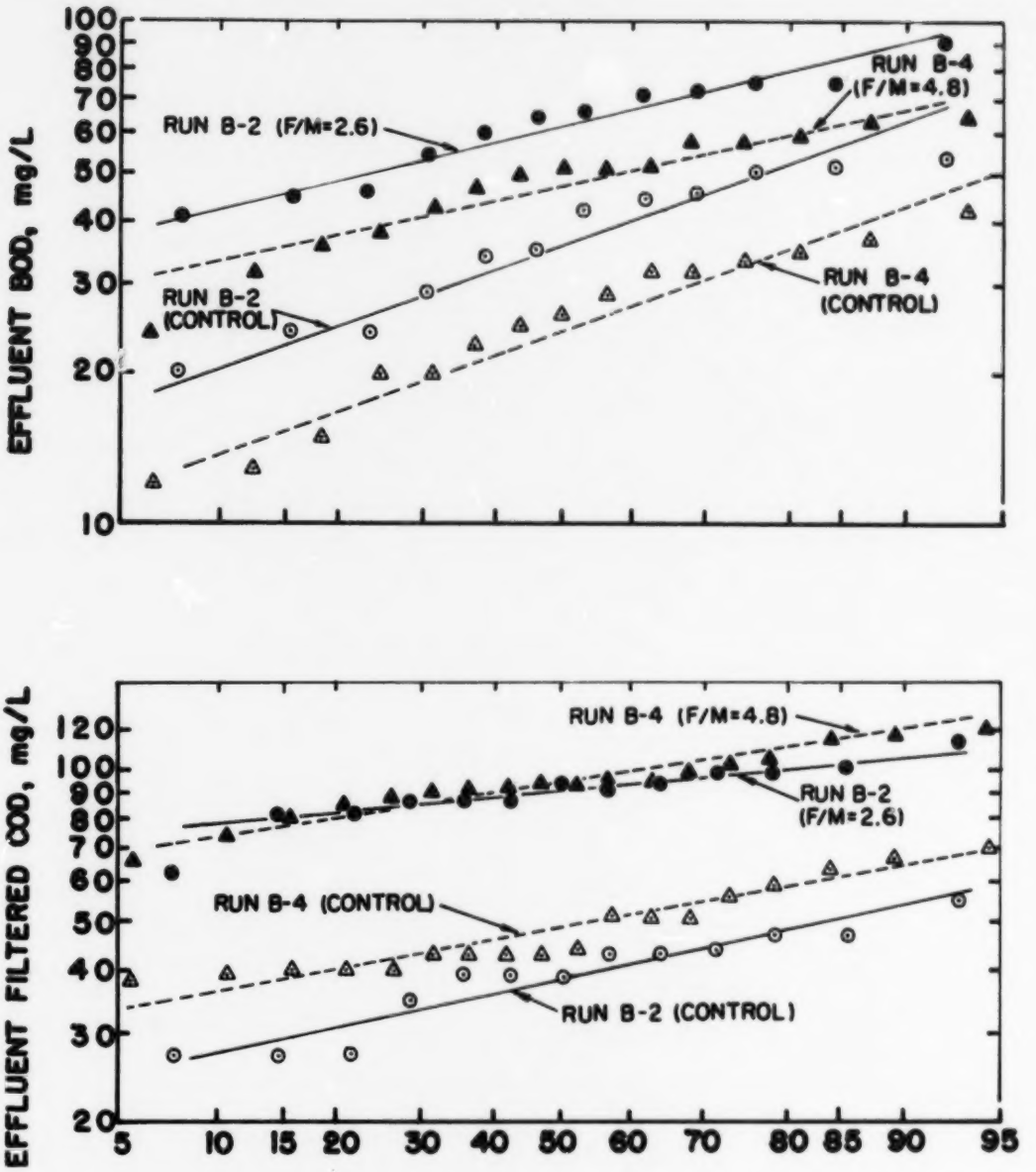


Figure 4

ACTIVATED SLUDGE TREATMENT AT HIGH LOAD INTENSITY

BENCH STUDY

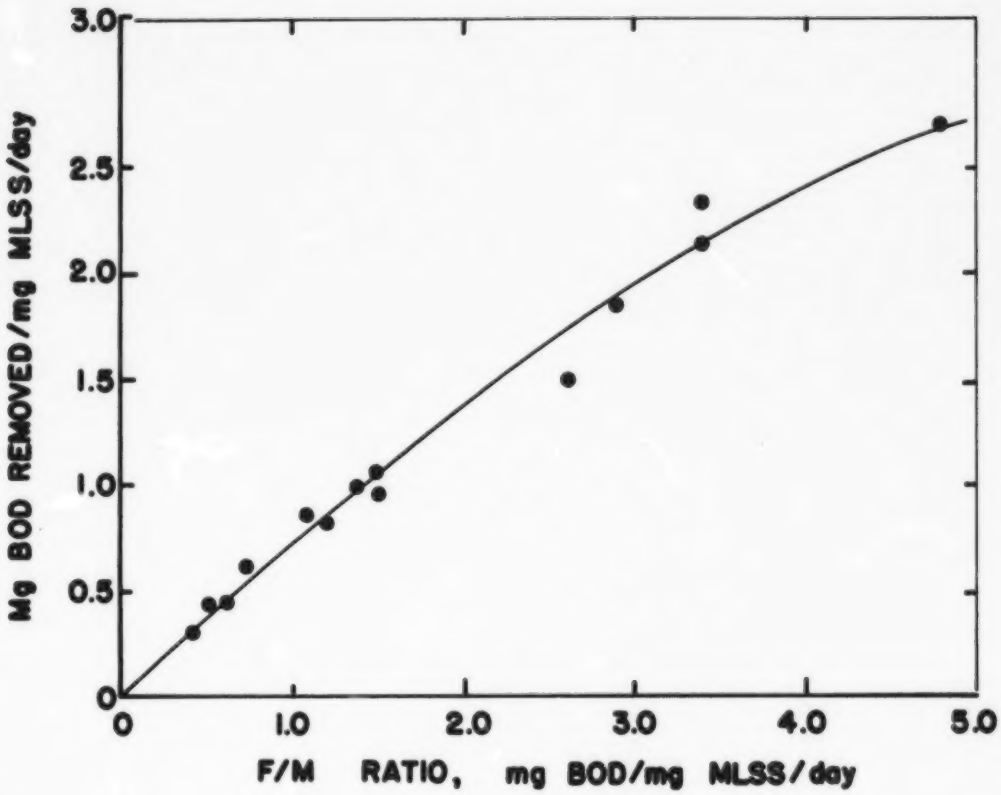


Figure 5

EFFECT OF LOAD INTENSITY ON SLUDGE ACTIVITY

BENCH STUDY

The observed relationship between F/M ratio and Sludge Volume Index presented in Figure 6 is also noteworthy. As reported by Haseltine[4], Stewart[5], and others, severe bulking occurred in an intermediate loading range, on either side of which this condition was not experienced. Also confirmed was the excellent treatment effected during bulking as long as the voluminous sludge was retained in the system.

Chemical Treatment

Foam Separation

Various workers have shown that surfactants can interfere with chemical coagulation[6]. It is also known that appreciable removal of such agents is affected by foam separation, an operation which, if beneficial, could be incorporated in the overall treatment process without difficulty or great cost.

Accordingly, experiments were conducted before routine jar testing was initiated to determine whether or not such treatment might be justified in working with Tucson sewage.

Nine liters of effluent from the bench activated-sludge control unit during the first test run were aerated at 19 cfh in a $4\frac{1}{2}$"-I.D. plastic cylinder. Foam was removed as it formed until the volume of collapsed foam was at least one per cent of the original sample. Effluent samples with and without foam removal were then jar tested in parallel.

From results typified in Figure 7, it was concluded that in this case such treatment prior to chemical coagulation could not be justified.

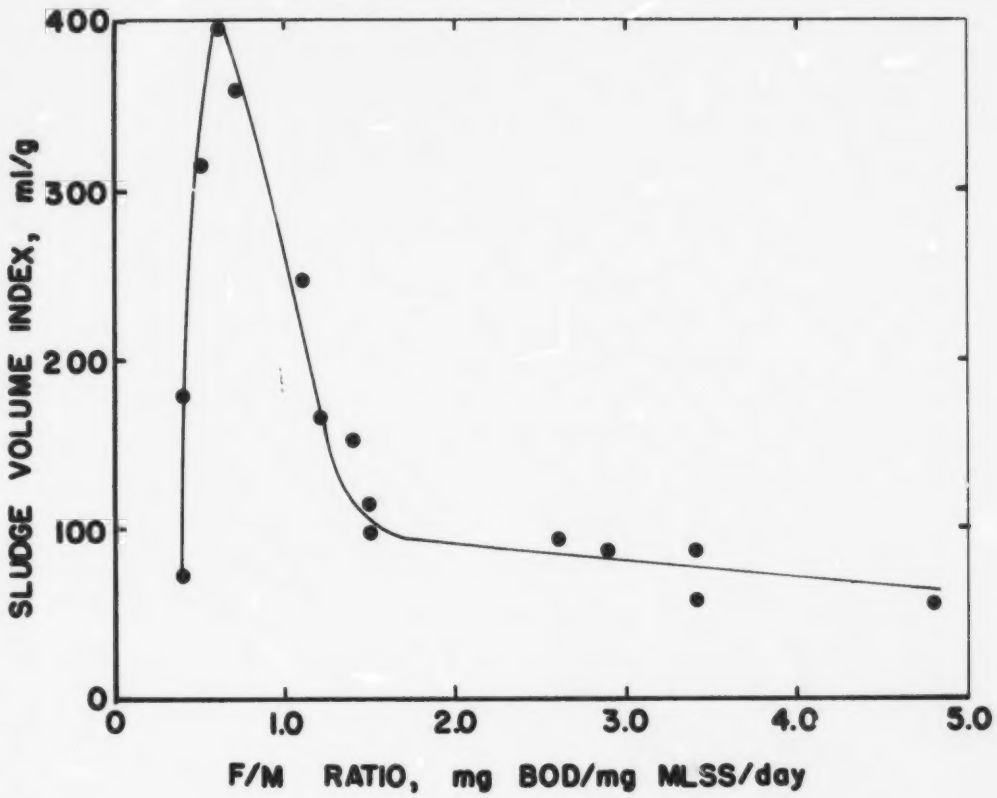


Figure 6

EFFECT OF LOAD INTENSITY ON SLUDGE VOLUME INDEX
BENCH STUDY

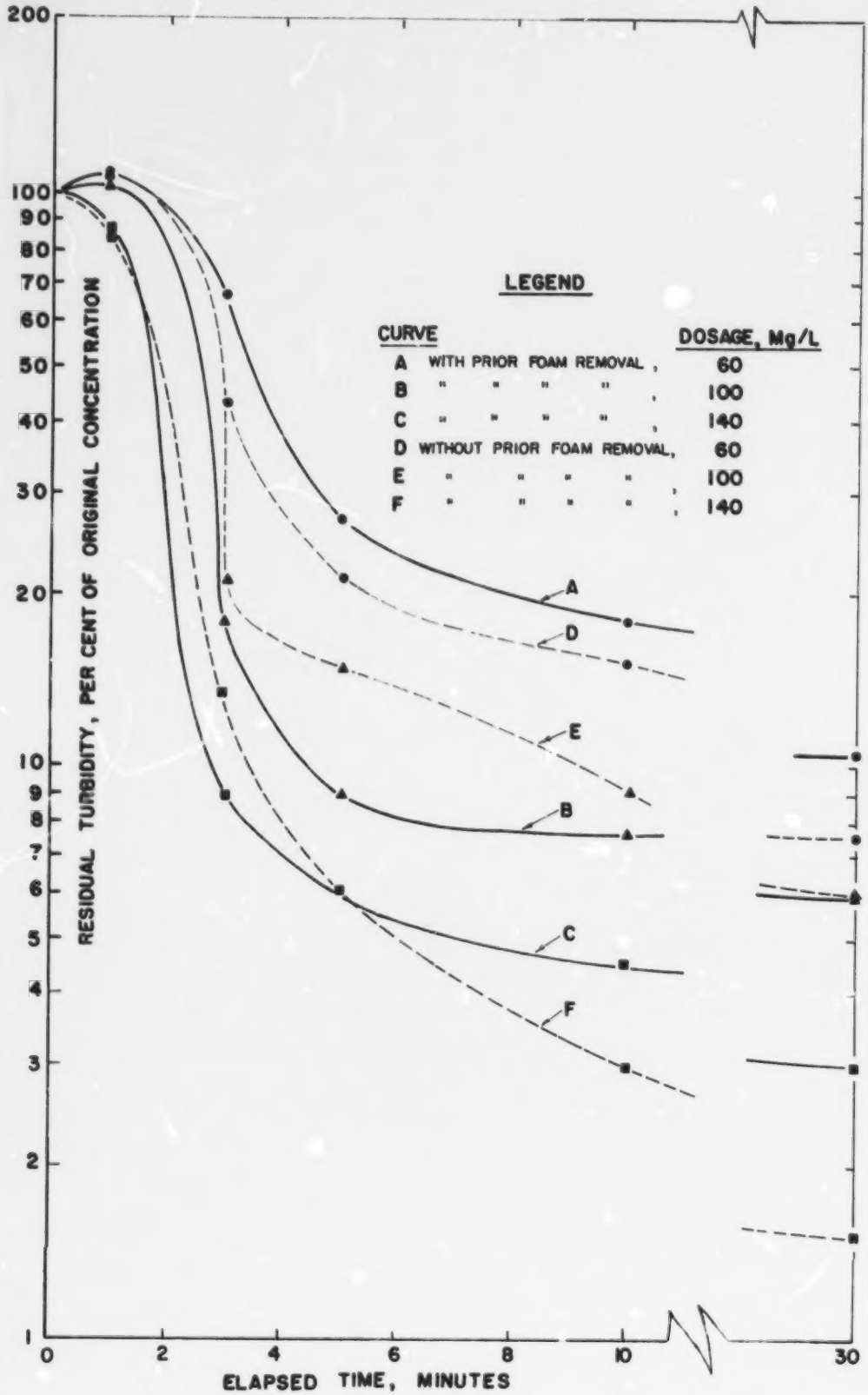


Figure 7
INFLUENCE OF FOAM SEPARATION ON COAGULATION
BENCH STUDY

Chemical Flocculation

Chemical flocculants evaluated included alum, ferric sulfate, ferric chloride, lime, and combinations of lime plus alum. In general, each of these was equally effective and the ultimate choice for further use in the pilot plant proved to be a matter of economics as much as anything else.

Complete daily jar testing of the effluent from each activated sludge system during each run was impractical. Instead, each effluent received the same dosage on a given day and the dosage was varied from day to day, repeating a range of dosages as often as possible during each run. Additional exploratory tests were conducted as time permitted. The results were evaluated in part by combining data so obtained for the various F/M groups noted in Table 4.

TABLE 4

Classification of Bench Runs by Load Intensity
For Analysis of Jar-Test Data

Group	Activated Sludge Bench System	F/M Ratio	Group	Activated Sludge Bench System	F/M Ratio
1	B-1-1	0.4	5	B-2-4	2.6
	B-1-2	0.5		B-2-5	2.9
	B-2-1	0.4		Group average	2.75
	Group average	0.43			
2	B-1-3	0.6	6	B-2-6	3.4
	B-1-4	0.7		B-2-7	3.4
	Group average	0.65		Group average	3.4
3	B-1-5	1.2	7	B-4-2	4.8
	B-2-2	1.1			
	Group average	1.15			
4	B-1-6	1.5			
	B-1-7	1.4			
	B-2-3	1.5			
	Group average	1.47			

Average results for alum, presented in Figure 8, show that the quantity of coagulant required to achieve a given effluent quality is a direct function of the first-stage load; the higher the F/M ratio, the greater the requisite dosage.

It is also clear that at high F/M ratios appreciable dosages of chemical had to be used to produce effluents comparable to those from the control units. Average effluent quality data for the conventionally loaded systems were:

BOD, mg/l	28
Suspended Solids, mg/l	28
Filtered COD, mg/l	47
Filtered TOC, mg/l	16
Turbidity, units	17

Examination of Figure 8 discloses that no difficulty was experienced in reducing supernatant BOD and suspended solids concentrations below those of the controls with 100 mg/l of alum. Turbidity removal was also no problem. As a matter of fact, lower dosages sufficed in this regard. But at high F/M ratios, at least 200 mg/l were needed to reduce the levels of filtered COD and TOC to those of the controls.

Interpretation of the absolute data of Figure 8 is complicated by differences in feed strength for the various experimental runs and the fact that some of the points are based on limited data. Further insight is provided by Figure 9 in which the results are presented in relative terms and it is quite apparent that increasing the alum dosage to 300 mg/l from 200 mg/l actually effected only nominal improvement in supernatant quality.

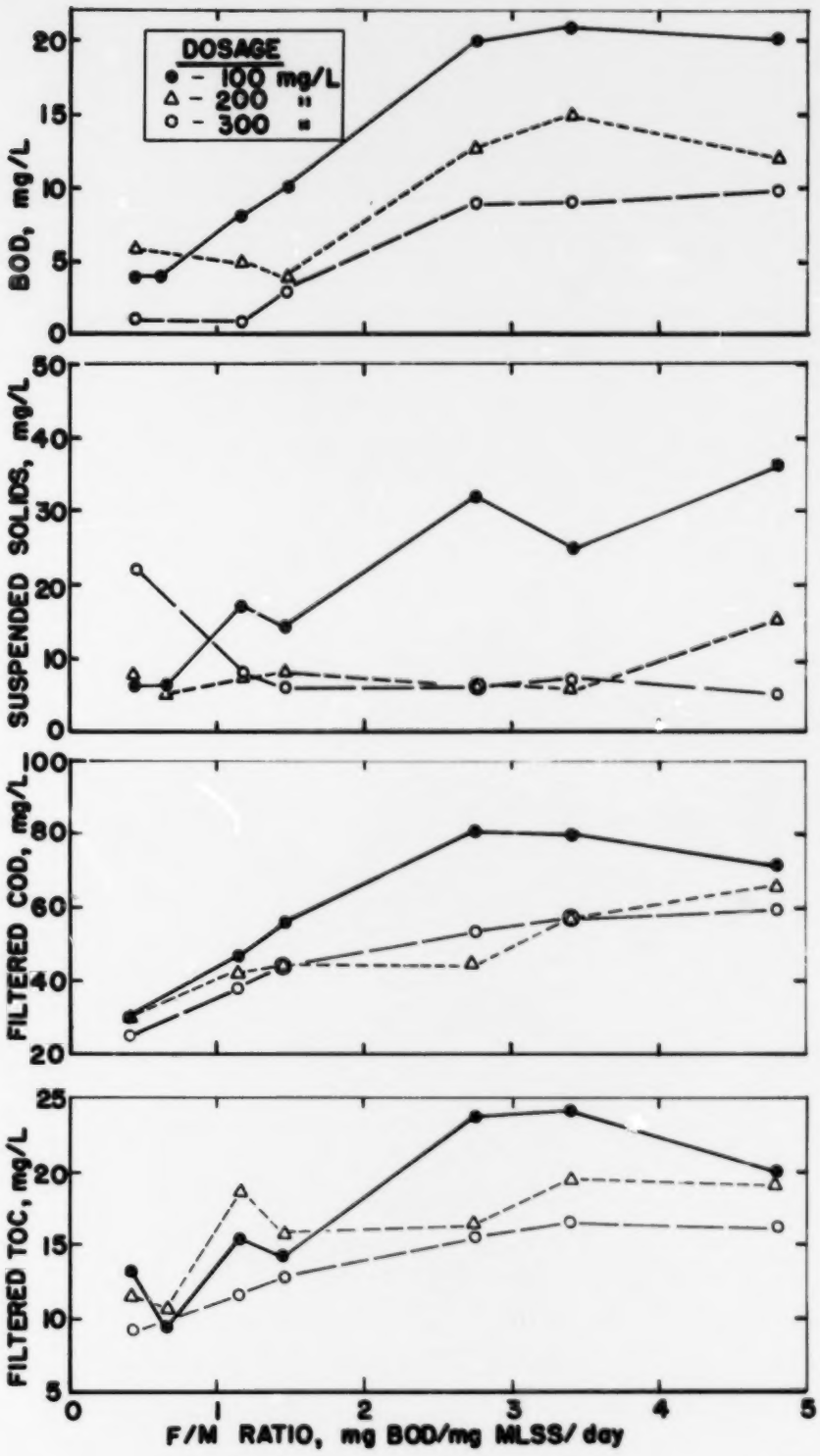


Figure 8

ALUM JAR-TEST QUALITY DATA, ABSOLUTE VALUES

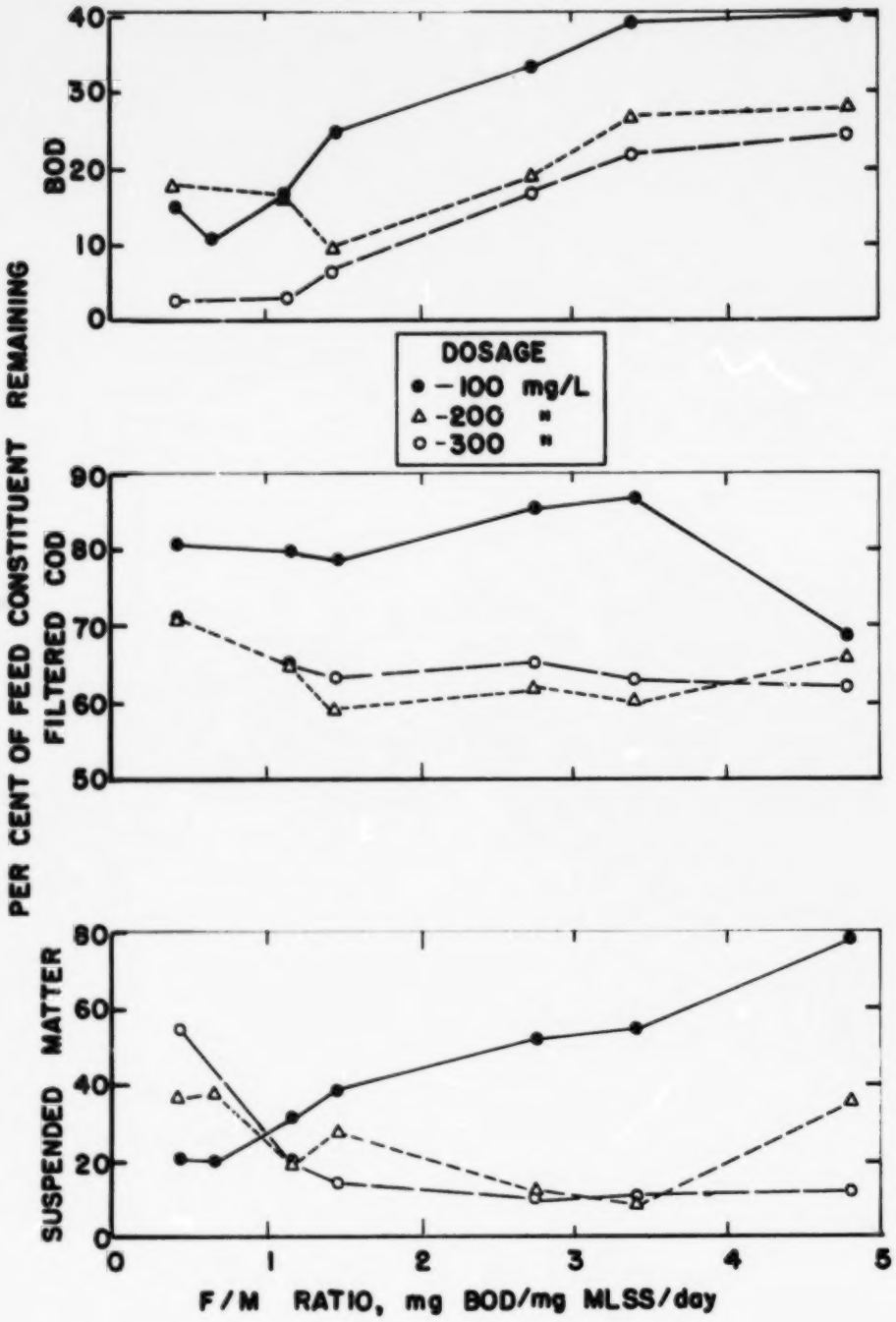


Figure 9

ALUM JAR-TEST QUALITY DATA, RELATIVE BASIS

As noted earlier, comparative tests with the other coagulants studied yielded similar findings. Supernatant quality data developed following 200-mg/l dosage of various coagulants to first-stage effluent produced at an F/M ratio of 4.8 are summarized in Table 5.

TABLE 5

Jar-Test Data: Supernatant Quality After Treatment of Highly-loaded Activated Sludge Effluent (F/M = 4.8) with 200 mg/l of Stated Coagulants

Chemical	BOD mg/l	Suspended Solids mg/l	Turbidity Units	Filtered COD mg/l	Filtered TOC mg/l	pH
Alum	13	20	12	63	16	7.2
Ferric sulfate	13	9	15	58	16	7.1
Lime	12	27	18	76	17	9.5
Lime + 20 mg/l alum	11	16	13	59	19	8.8

The effectiveness of the combination of lime plus a small quantity of alum was disclosed by numerous tests. Addition of alum as a flocculant aid improved removal of pin-point floc which resulted after dosage with lime alone. It was also determined that for maximum effect the lime should be added first.

Addition of Waste Activated Sludge to Chemical Treatment

The relatively thin excess activated sludge generated by first-stage treatment must be disposed of in some manner. Conceivably, this material could be wasted to the second-stage for mixture and concentration with the waste chemical sludge. Second-stage addition of such solids in combination with chemical flocculants might also provide added treatment.

However, addition of MLSS in the concentrations which would exist in practice (25 to 100 mg/l) during jar testing in Run B-4 failed to demonstrate any treatment advantage for such a scheme. On the contrary, some deterioration of supernatant quality was noted in the case of each of the coagulants studied. Typical data for the combination of lime and alum are shown in Table 6.

TABLE 6

Jar-Test Data: Supernatant Quality after Treatment of Highly-loaded Activated Sludge Effluent (F/M = 4.8) with 200 mg/l Lime + 20 mg/l Alum + Stated Concentrations of Activated Sludge

MLSS added mg dry solids/l	BOD mg/l	Filtered		Suspended Solids mg/l	Turbidity Units
		mg/l	mg/l		
0	18	79	20	12	7
20	18	79	22	23	11
40	15	79	23	23	10
60	21	79	21	30	11
80	16	83	22	24	10
100	15	91	24	27	12

It should not be inferred, however, that discharge of excess activated sludge to second-stage treatment is entirely impracticable. Under circumstances where optimum final effluent quality is not required, the ability to thicken waste activated sludge in this manner would be a significant advantage.

Solids-Contact Treatment

Design and fabrication of pilot plant units for later operation were undertaken toward the end of the laboratory studies and inasmuch as use of a solids-contact chemical treatment unit was visualized, this type of treatment was evaluated briefly during Run B-4.

Jar-test procedure entailed comparison of supernatant quality following a single chemical dosage with that resulting from treatment in the presence of previously precipitated solids provided by sludge return. In the latter case, solids were accumulated through five successive cycles of dosage, mixing, and settling, supernatant being discarded at the end of each of the first four cycles.

As shown in Table 7, significant improvement of supernatant quality attributable to solids-contact treatment was limited to suspended solids and turbidity. Differences in the other quality criteria noted are inconclusive, although separate research confirms the observed consistent additional phosphate removal.

TABLE 7

Jar-Test Data: Average Supernatant Quality After Solids-contact Treatment of Highly-loaded Activated Sludge Effluent (F/M = 4.8)

Chemical Treatment	Alum 100 mg/l		Ferric Sulfate 100 mg/l		Lime 200 mg/l + Alum 20 mg/l	
Number of sludge returns	0	4	0	4	0	4
Number of jar tests	3	3	3	3	1	1
BOD, mg/l	25	23	18	17	14	17
COD, filtered, mg/l	58	60	64	64	72	76
TOC, filtered, mg/l	18	17	22	21	19	19
Suspended solids, mg/l	33	20	26	11	17	3
Turbidity, units	25	15	20	18	14	15
Total phosphate, mg/l as PO_4^{3-}	17	15	13	12	8	5

Phosphate Removal

The phosphorous content of sewage effluents exerts an important fertilizing effect upon the receiving waters, stimulating plant and animal growth objectionably in some instances. Its removal by conventional secondary treatment is limited to about 50 per cent and the technical feasibility of its further reduction to low concentrations by chemical treatment has been demonstrated by several investigators[7]. In view of the relatively large chemical dosages required to reduce other effluent constituents to desirable levels, it seemed likely that significant phosphorous removal might be realized as an important added advantage of the treatment process under study.

Accordingly, residual total phosphate as a function of dosage for several coagulants was determined during Run B-4 on effluent from the activated-sludge unit loaded at an F/M ratio of 4.8.

The findings, summarized in Figure 10, show that phosphate reduction was similar for each of the coagulants studied, and further that 200 to 250 mg/l were required to lower the original total phosphate-ion content of about 35 mg/l to 3 mg/l (equivalent to 1 mg/l as P).

Discussion

While improved pilot-plant chemical treatment over that obtained in the laboratory could be expected due to limitations of the latter associated largely with the jar tests, it appeared at this point that operation of the activated sludge system at high loads necessitated a

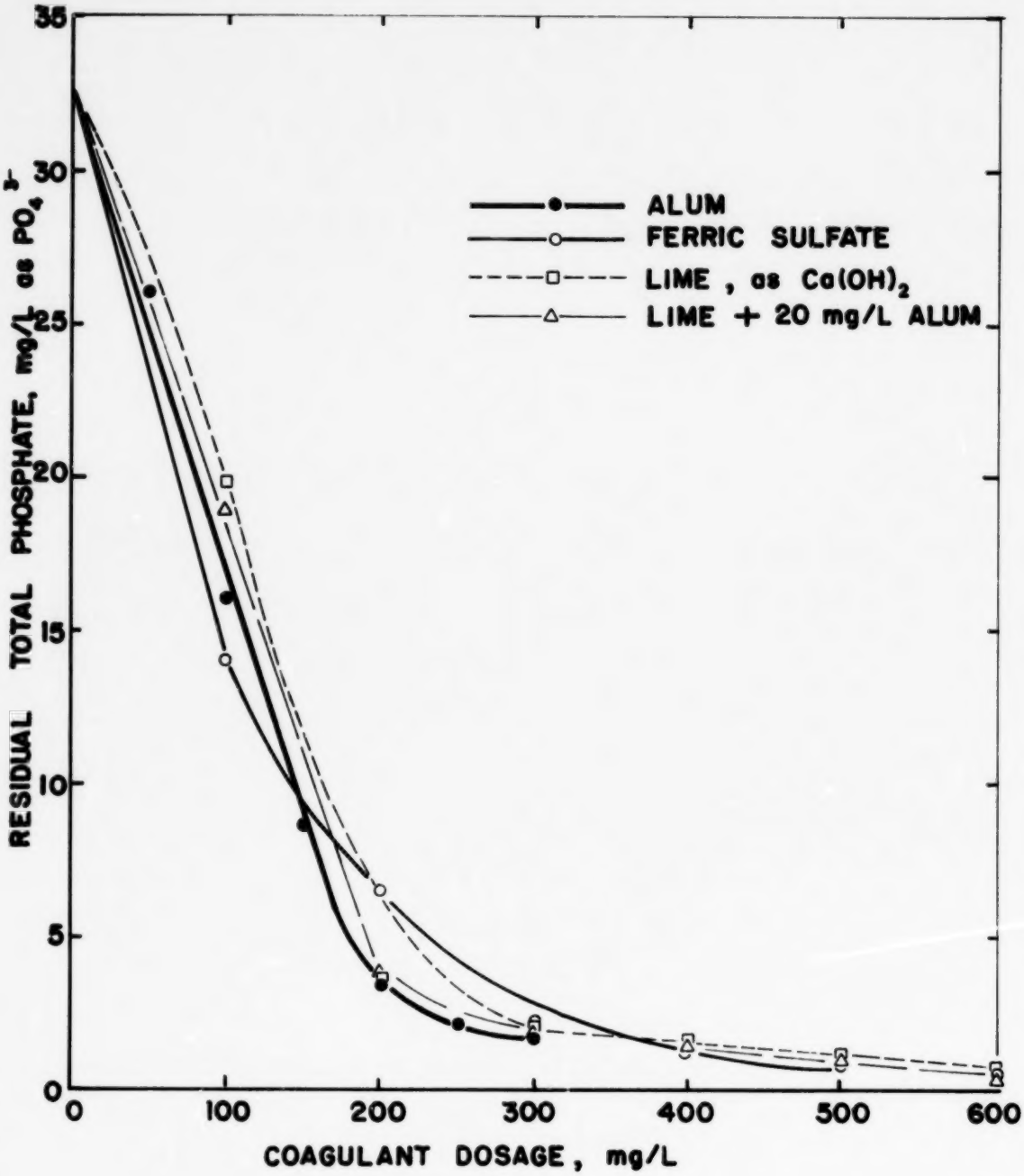


Figure 10

PHOSPHATE REMOVAL BY VARIOUS COAGULANTS

subsequent minimum dosage of 200 mg/l of any of the chemicals studied to obtain the final effluent quality desired. In addition, chemical treatment at this level offered the important concomitant advantage of reducing residual total phosphate to a low concentration.

Phosphate removal from activated sludge sewage effluent by chemical techniques has been investigated extensively by Wuhrmann and his co-workers in Switzerland[8]. From laboratory and pilot plant studies using a trivalent iron salt ($FeCl_3$) alone, lime alone at high pH, and lime plus a small concentration of Fe^{3+}, they concluded that dosage with lime to pH ±8.8 (100 to 120 mg/l as $Ca(OH)_2$ in their case) plus 10 mg/l Fe^{3+} was effective and the most economic solution on the basis of the volume and treatability of waste sludge, as well as chemical cost.

With this added support, the combination of 200 mg/l lime plus 20 mg/l alum was therefore selected for further study in the pilot plant.

PART II: PILOT PLANT STUDY

Following development and analysis of the laboratory data, a 10-gpm pilot plant was constructed and installed at the municipal sewage treatment plant. Design criteria at average flow included a minimum first-stage aeration time of one hour and clarifier overflow rates of 700 gpd/ft^2 in both stages.

Selection of the 1-hr aeration time was entirely arbitrary and was influenced by the necessity for adequate contact time at peak flow

rates. With both aeration volume and flow fixed, the upper limit of the range of BOD-to-solids loading which could be realized was then imposed by the minimum practicable MLSS concentration. The latter was selected as being about 500 mg/l and it was decided to first study performance at an F/M ratio of about 4.0 and then at approximately half this loading.

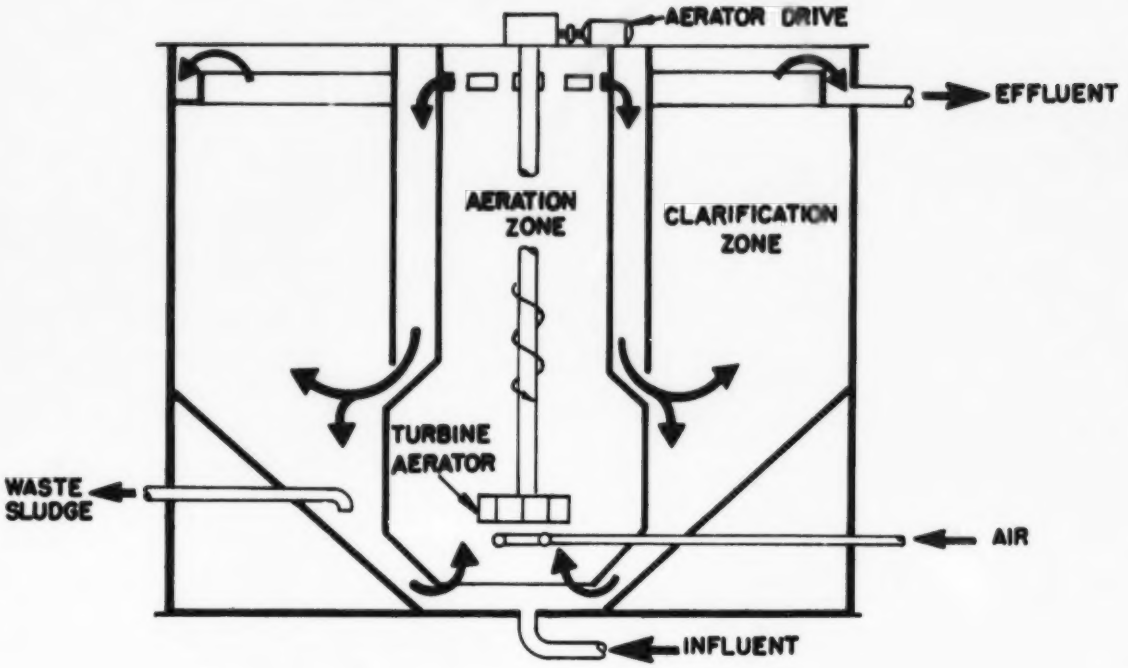
Apparatus and Procedure

The two main pilot plant components, an AERO-ACCELATOR activated sludge unit and a CYCLATOR solids-contact chemical treatment unit, were located together with the requisite ancillary equipment adjacent to one of the primary clarifiers at the sewage works. Both units, shown schematically in Figure 11, are widely used in the treatment of domestic and industrial wastewaters.

In the AERO-ACCELATOR unit, the processes of completely-mixed aeration, sludge return, and liquid-solids separation are effected in a single compartmented tank. The 6.5-ft diameter x 7-ft SLD pilot unit had a mixed-liquor volume of 600 gal and a clarification area of 26.1 ft^2, making the aeration period one hour at average flow and the hydraulic loading in the clarification section 550 gpd/ft^2. Oxygenation and mixing were supplied by a turbine aerator and blower.

Mixing of chemicals and waste, flocculation, and solids separation are combined in the CYCLATOR clarifier. This 6-ft diameter x 4-ft SLD unit had 21.2 ft^2 of clarification area, making the overflow rate at nominal capacity 680 gpd/ft^2.

A. AERO-ACCELATOR ACTIVATED SLUDGE UNIT



B. CYCLATOR CHEMICAL TREATMENT UNIT

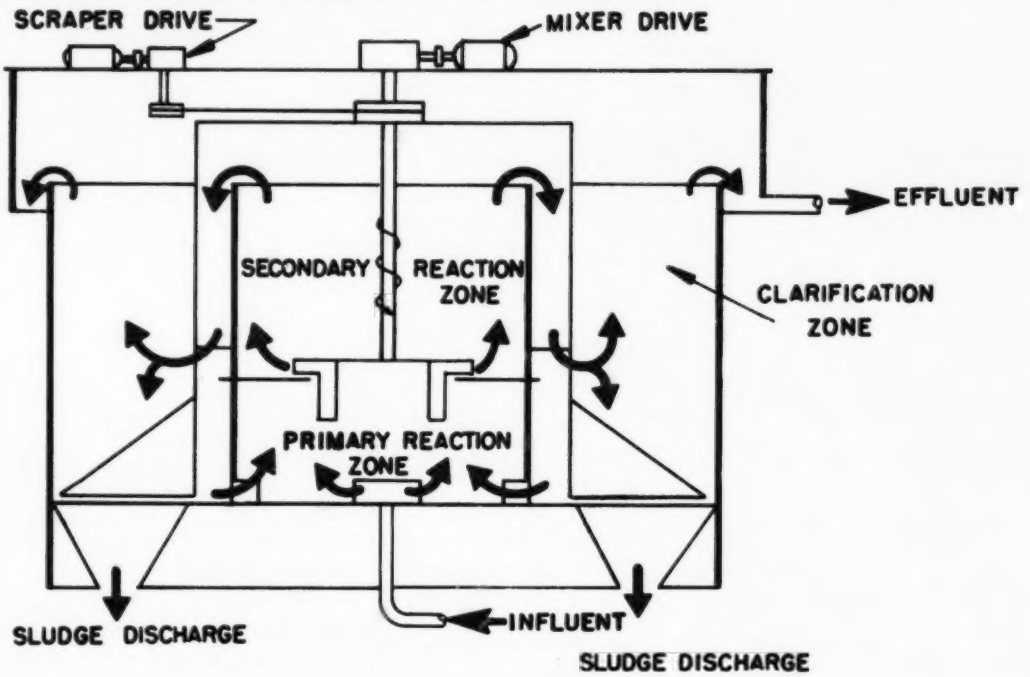


Figure 11

PRINCIPAL PILOT PLANT UNITS

Figure 12 shows the complete pilot plant. Settled sewage from the launder of a primary clarifier was pumped at controlled rates to the activated sludge unit (1) which discharged through a rapid-mix tank (2) to the chemical treatment unit (3). The external mix tank which was the point of lime application was inserted in the flowsheet when it was determined that reaction time in the small CYCLATOR clarifier was insufficient. Alum was fed to the primary reaction zone of the CYCLATOR.

Pilot plant throughput was varied in parallel with the main plant flow by means of controls tied into the pneumatic flow measurement system of the full-scale plant. The latter provided a proportional air signal against which a pressure differential from an orifice (4) in the pilot plant influent line was paced, using instrumentation (5, 6, and 7) which automatically modulated a flow-control valve (8).

The same instrumentation enabled flow-proportioned feed of lime slurry and alum solution to maintain constant dosages. Solution-type feeders (9) and chemical storage tanks (10 and 11) were provided.

Biological and chemical sludge concentrations were adjusted as desired by periodic wasting in proportion to flow through timer-controlled (12) air-operated blowoff valves (13). Both types of waste sludge were collected to permit determination of volume and solids content.

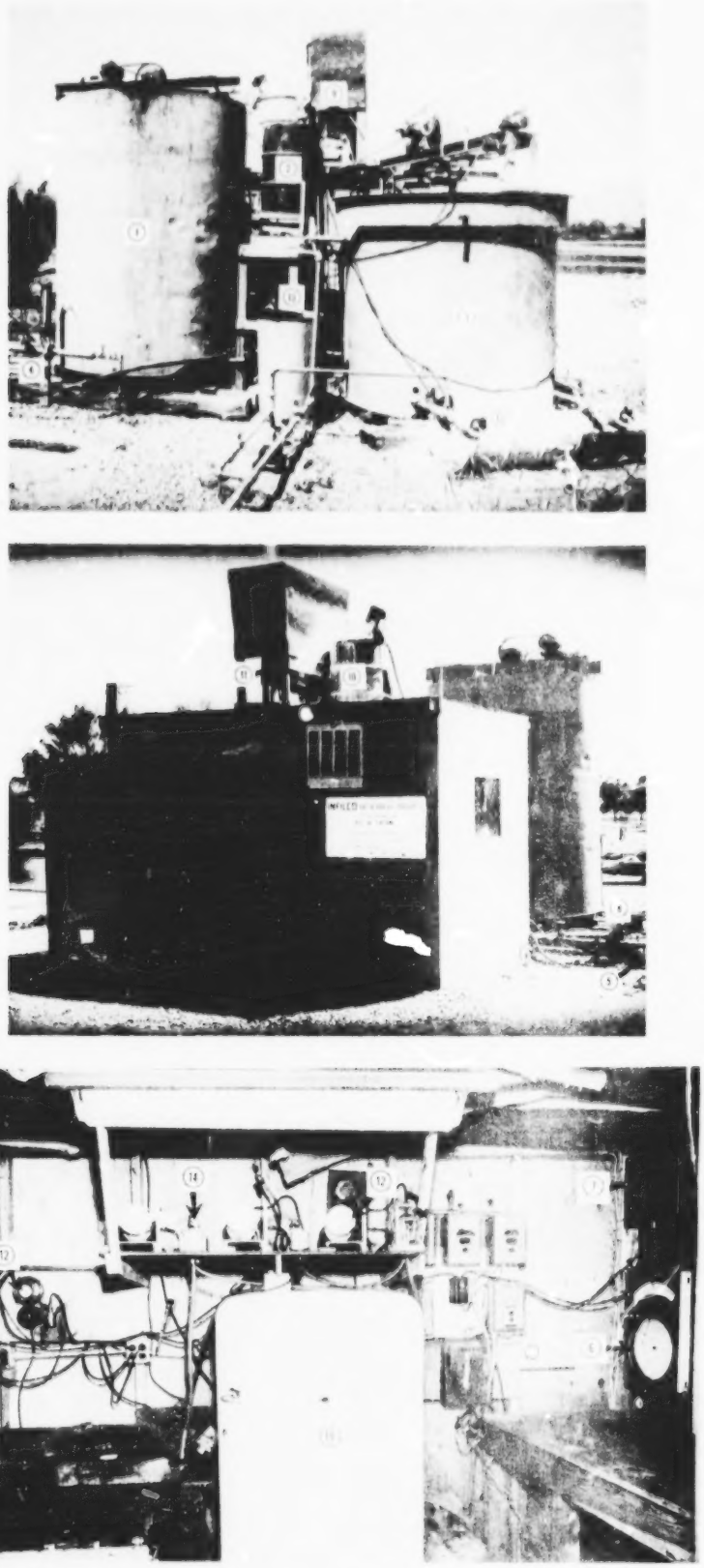


Figure 12.

PILOT-PLANT ACTIVATED SLUDGE AND CHEMICAL TREATMENT SYSTEMS

Samples of the feed and effluents from the two pilot units were collected daily. Flow-proportioned 24-hr composites were collected automatically by systems (14) which sampled each stream at intervals of a few minutes. The main-plant activated-sludge effluent prior to chlorination was sampled manually every two hours in proportion to flow to accumulate 24-hr composites. All samples were refrigerated (15) during collection.

The F/M load ratio for the activated sludge pilot unit was controlled by wasting MLSS as required from knowledge of the feed flow and approximate BOD. Mixed-liquor settleability, SVI, and microscopic characteristics were determined twice daily in an effort to maintain close operational control.

Oxygen demand was evaluated through uptake rate determinations made in the aeration compartment of the activated sludge unit. The air supply was shut off and, with influent flow and mixing continuing, the rate of decrease in mixed-liquor DO concentration (normally carried at several mg/l) was followed with an oxygen probe. Measurements of this type were made twice daily, once in the early morning during minimum flow and again in the late afternoon at the time of peak loading.

Results and Discussion

Flow through the main plant averaged close to 7 mgd during the period of study. Pilot plant daily flow, Figure 13, was very close to that desired. Average throughput rates of 10.4 and 10.7 gpm were obtained for each of the respective runs. Variability for each series

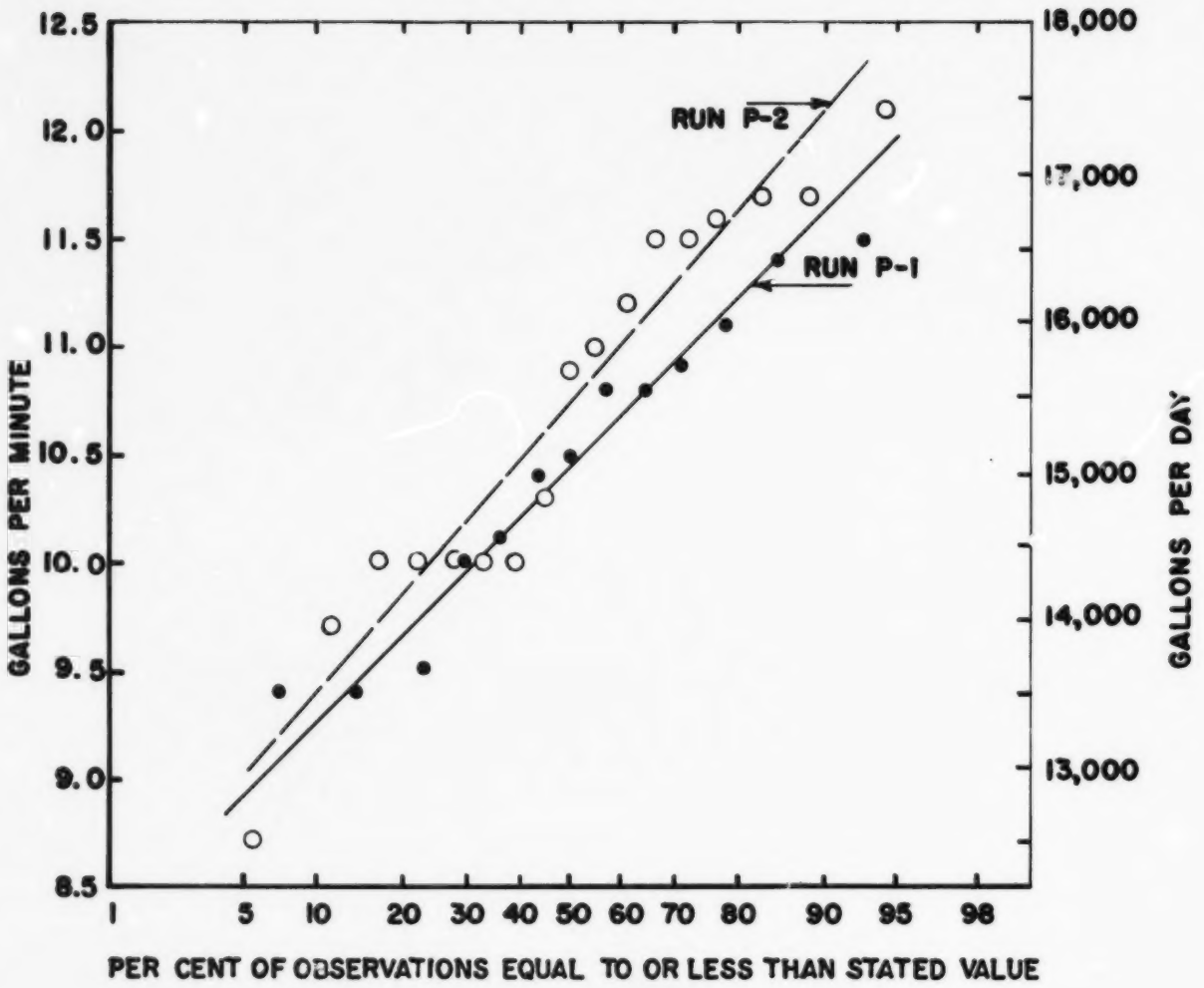


Figure 13

DAILY PILOT PLANT FLOW RATES

was similar and, considering the nature of the flow control system, reflected differences in the main-plant daily flow. Peak and minimum flow rates approximated 150 and 50 per cent of the daily average, respectively.

Performance of the pilot and main plants is summarized in Tables 8 and 9, together with Figures 14 and 15. Residual BOD, suspended solids, and especially total phosphate were appreciably lower in the pilot plant effluent; however, turbidity, total and filtered COD, and filtered TOC were higher. Inasmuch as the observed differences, although small in some cases, are greater than any probable sampling error, it is evident that the pilot plant as operated during this investigation was somewhat less effective than the main plant in certain respects.

Several samples during each run were analyzed for their organic nitrogen, ammonia, and nitrate content. The results confirmed the expectation of little or no nitrification. Organic nitrogen was reduced only a few mg/l below the feed concentration of about 7 mg/l. Pilot plant effluent for both series contained some 20 mg/l of ammonia nitrogen and no nitrate. Main-plant effluent averaged 2 mg/l organic nitrogen, 17 mg/l ammonia nitrogen, and virtually no nitrate nitrogen.

Analyses for ABS indicated that about half of the primary effluent content of 8 mg/l was removed by the main plant, contrasted with only 20 per cent reduction through the pilot plant. However, the small number of determinations made limits the significance of this observation.

TABLE 8

Average Pilot Plant Performance Data

Item	Run P-1 F/M = 4.1	Run P-2 F/M = 2.7
Duration of test, days	12	13
Flow, gpm	10.4	10.7
gpd	15000	15400
Liquid temperature, °C	32	31
Activated Sludge Unit		
Aeration time, minutes	58	56
Overflow rate, gpd/ft^2	570	590
MLSS, concentration, mg/l	520	780
30-minute settleability, ml/l	86	140
sludge volume index, ml/g	170	180
Waste sludge, volume, gpd	756	640
solids content, % by weight	0.052	0.073
solids, lb/day	3.3	3.9
mg solids/mg BOD removed/day	0.6	0.5
Oxygen, uptake rate, mg/l/hr	43	45
uptake rate, mg/hr/gram MLSS	83	58
requirement, mg/mg BOD removed	0.8	0.7
Chemical Treatment Unit*		
Slurry-pool detention period, minutes	47	46
Overflow rate, gpd/ft^2	710	725
Waste sludge, volume, gpd	153	160
volume, % of throughput	1.0	1.0
solids content, % by weight	3.8	4.1
solids, lb/day	48	54

*Chemical dosage: 200 mg/l lime + 20 mg/l alum during both runs.

TABLE 9

Average Performance Data: Pilot Plant Versus Main Plant*

Item	Pilot Plant F/M = 4.1			Main Plant F/M = 0.2		Pilot Plant F/M = 2.7			Main Plant F/M = 0.2	
	Inf	Eff	% Red'n	Eff	% Red'n	Inf	Eff	% Red'n	Eff	% Red'n
BOD, mg/l	74	18	76	46	38	88	15	83	47	47
Suspended Solids, mg/l	47	9	81	16	66	52	12	77	22	58
Total COD, mg/l	189	68	64	63	67	197	72	64	64	67
Filtered COD, mg/l	102	-	-	42	59	113	60	47	42	63
Filtered TOC, mg/l	30	20	33	14	53	31	18	42	13	58
Turbidity, units	128	25	80	22	83	140	29	79	22	84
Total Phosphate, mg/l	28	1.8	94	26	7	27	2.3	92	24	11
pH, units	7.5	9.4	-	7.6	-	7.8	8.9	-	7.8	-

*Second-stage pilot plant chemical dosage 200 mg/l lime + 20 mg/l alum during both test runs.

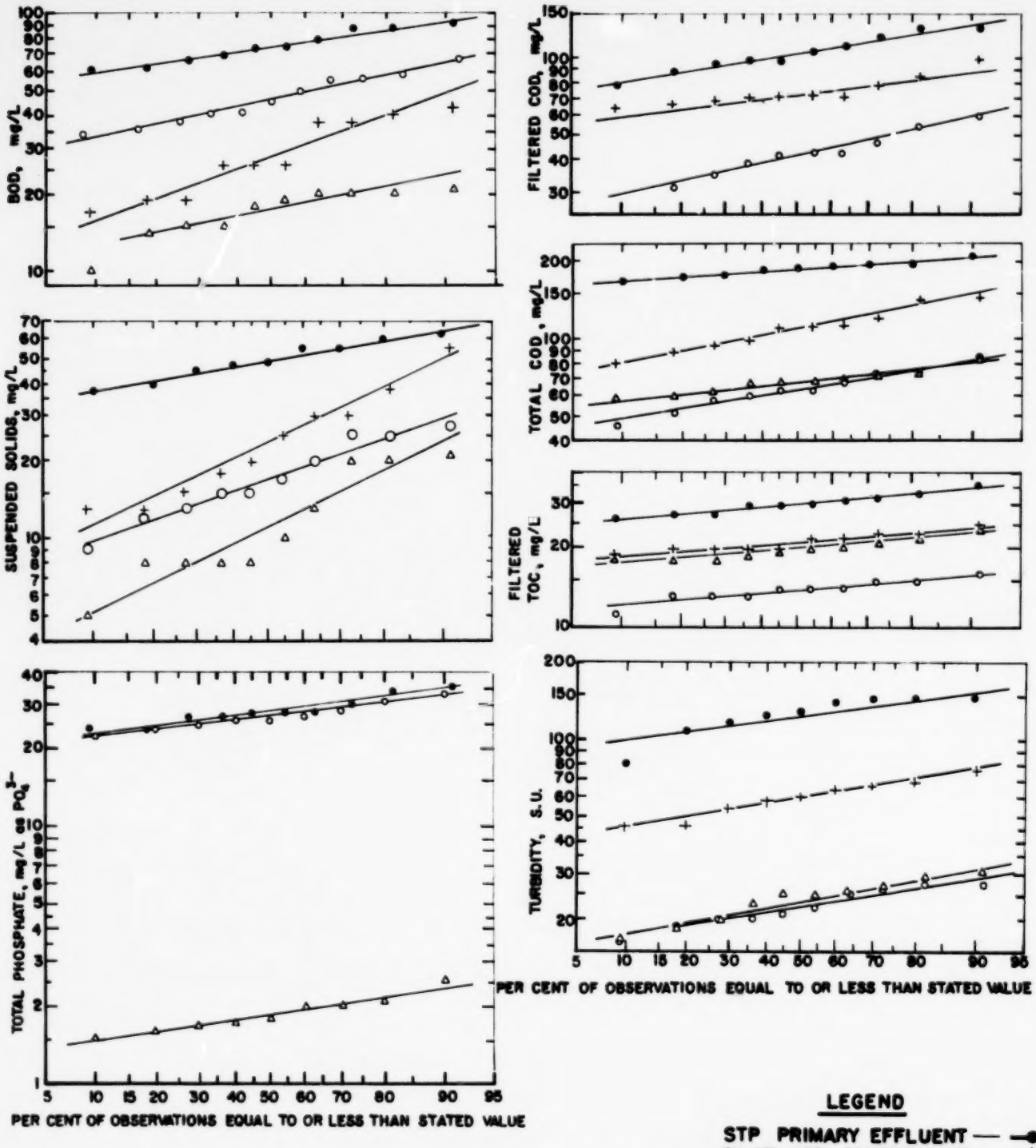

BOD, mg/L
FILTERED COD, mg/L
SUSPENDED SOLIDS, mg/L
TOTAL COD, mg/L
FILTERED TOC, mg/L
TOTAL PHOSPHATE, mg/L as PO_4^{3-}
TURBIDITY, S.U.
PER CENT OF OBSERVATIONS EQUAL TO OR LESS THAN STATED VALUE
PER CENT OF OBSERVATIONS EQUAL TO OR LESS THAN STATED VALUE
LEGEND
STP PRIMARY EFFLUENT
PILOT ACT. SLUDGE UNIT
PILOT CHEMICAL UNIT
FULL-SCALE STP

Figure 14

EFFLUENT QUALITY, RUN P-1

PILOT AND FULL-SCALE TREATMENT SYSTEMS

FIRST-STAGE PILOT-PLANT F/M RATIO 4.1

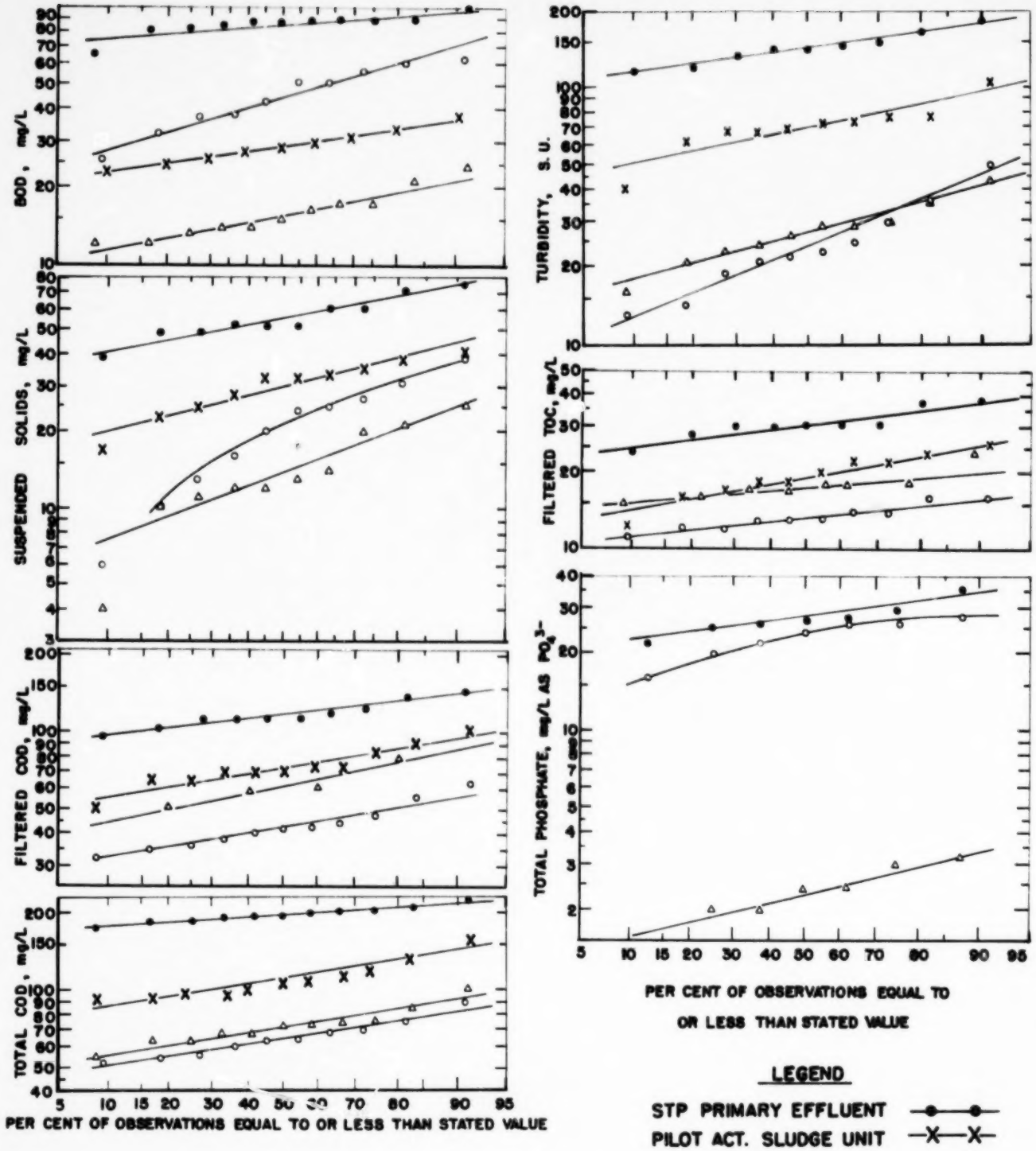


Figure 15

EFFLUENT QUALITY, RUN EFFLUENT QUALITY, RUN P-2
PILOT AND FULL-SCALE TREATMENT SYSTEMS
FIRST-STAGE PILOT-PLANT F/M RATIO-2.7

Effluent quality for the two pilot plant runs was essentially equivalent. On the basis of suspended matter, filtered COD and filtered TOC, this quality equaled or bettered the laboratory findings (see Table 5). But residual BOD and turbidity were slightly higher.

Examination of the cumulative frequency distribution curves of Figures 14 and 15 reveals no significant difference in the dispersion of data for each quality parameter, indicating no advantage in terms of process stability for any of the systems studied. A corollary observation is that performance of the pilot plant with its high first-stage loading was as stable as that of the main plant.

Performance of the activated sludge unit for the two pilot runs and comparisons with similarly loaded bench units are given in Table 10. As before, the geometric mean is used as the best measure of central tendency. Except for suspended solids, removables for the pilot plant series were about the same in spite of the difference in average loadings. Effluent at the lower loading contained appreciably more suspended matter, contrary to expectation.

A possible explanation for the unexpected effluent equivalency lies in the inability to closely control F/M ratios during each run of limited duration. Because of the low feed strength, it was necessary to hold MLSS concentrations in the pilot activated sludge unit at relatively low and quite sensitive levels and this was achieved with only fair success. It is evident from Figure 16 that although average load intensity for each run was roughly as desired, considerable variation occurred in fact.

TABLE 10

Average Performance: Pilot Versus Laboratory Activated-Sludge Units

Item	Pilot Unit F/M = 4.1			Bench Unit F/M = 4.8			Pilot Unit F/M = 2.7			Bench Unit F/M = 2.6		
	Inf	Eff	% Red'n	Inf	Eff	% Red'n	Inf	Eff	% Red'n	Inf	Eff	% Red'n
BOD, mg/l	74	26	65	113	51	55	88	28	58	158	66	58
Suspended Solids, mg/l	47	22	53	73	47	35	52	32	39	102	56	45
Filtered COD, mg/l	102	71	30	143	94	34	113	69	39	154	94	39
Filtered TOC, mg/l	30	21	30	46	30	34	30	19	37	-	28	-
Turbidity, units	128	60	53	42	35	17	140	71	49	135	58	57

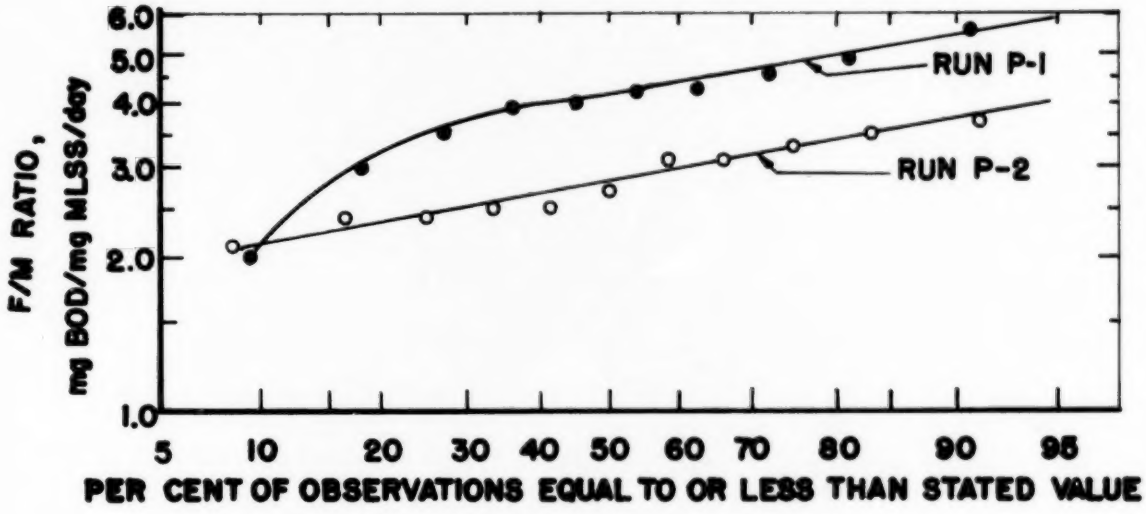


Figure 16

FIRST-STAGE F/M RATIOS, PILOT-PL/ . RUNS

Direct comparison of absolute effluent quality for the pilot unit and similarly loaded bench units is not possible in view of marked differences in feed strength. However, percentage reductions agree reasonably well.

BOD removal across the activated sludge unit amounted to 2.4 and 1.8 mg/mg MLSS/day for operation at F/M ratios of 4.1 and 2.7, respectively. Of interest is the excellent agreement of these values with performance of the bench units (see Figure 5).

The oxygen data given in Table 8 appear reasonable and in the proper relationship. The requirements noted are believed to be conservative because uptake rates were not obtained over the complete cycle of load variation each day. Measured as they were daily in the morning and afternoon, the lower oxygen demand which undoubtedly prevailed during low-flow periods at night was missed.

Excess sludge produced during activated sludge treatment, amounting to 0.5-0.6 lb of solids/lb of BOD removed/day, is somewhat less than expected - perhaps reflecting the low influent suspended solids.

Performance of the chemical unit and pertinent laboratory jar-test data are shown in Table 11. As expected, influent BOD, suspended matter, and turbidity were reduced appreciably but almost no additional removal of the already low filtered COD and TOC resulted. Phosphate was reduced significantly to residual concentrations averaging no more than 10 per cent of the influent content.

TABLE 11

Average Performance Data: Pilot Chemical Unit Versus Laboratory Jar Tests*

Item	Pilot Unit F/M = 4.1			Jar Tests F/M = 4.8			Pilot Unit F/M = 2.7		
	Inf	Eff	% Red'n	Inf	Eff	% Red'n	Inf	Eff	% Red'n
BOD, mg/l	26	18	31	44	13	70	28	15	46
Suspended Solids, mg/l	22	9	59	39	16	59	32	12	62
Filtered COD, mg/l	-	-	-	82	66	20	69	60	13
Filtered TOC, mg/l	21	20	5	28	19	32	19	18	5
Turbidity, units	60	25	58	41	15	63	71	29	59
Total Phosphate, mg/l	27	1.8	93	37	4.4	88	24	2.3	90

*Chemical dosage: 200 mg/l lime + 20 mg/l for both.

Jar testing at the same chemical dosages during the pilot plant runs yielded supernatant of consistently superior visual quality, and it is felt that design and functioning of the pilot chemical unit were less than optimum.

Production of excess chemical sludge, amounting to one per cent of the pilot plant throughput by volume, was about as anticipated from experience with this type of treatment.

Filterability of the various waste sludges produced in the overall treatment system was studied briefly. It was found that the chemical sludge dewatered rapidly without addition of conditioning agents and that inclusion of the small fraction of waste activated sludge solids had little effect on filterability. Combination of the process sludges with raw primary sludge resulted in a mixture which dewatered better than primary sludge alone, indicating the feasibility of vacuum filtration of the total sludge mixture prior to ultimate disposal.

GENERAL DISCUSSION

It is unfortunate that the period of pilot plant study coincided with the local summer rainy season. Feed strength was inordinately low, undoubtedly reflecting considerable dilution by infiltration. The 80-mg/l feed BOD was little more than a half that of normal settled domestic sewage and the suspended solids content of only 50 mg/l was even less than half that of normal primary effluent. Activated sludge treatment of this waste, even at high loading, left very little for chemical treatment to do.

Certainly, use of the main plant as a control was largely invalidated because it did not function at conventional activated sludge loading. Its average BOD-to-solids load ratio of 0.2 moves it into the range of extended-aeration treatment, rather than conventional, and probably accounts for its relatively high effluent BOD of nearly 50 mg/l prior to chlorination. It is likely that nitrification was incipient in the effluent and proceeded during the BOD tests, elevating the observed values.

It is concluded that the integrated system is capable of producing effluent of as good or better quality than that from the traditional activated sludge process.

From a practical point of view there appears to be no advantage to use of a very high first-stage loading. In the treatment of domestic sewage, at least, it is felt that a minimum aeration time of one hour at average design flow should be provided. Operation at an MLSS concentration of 2000 mg/l is entirely practicable and for a typical settled-sewage feed BOD of 150 mg/l the resultant load intensity is 1.8. Removal of

soluble organics is enhanced at this loading and BOD removal across the first-stage can be expected to increase from the roughly 60 per cent observed at the maximum load studied to about 70 per cent. The additional BOD removal of course necessitates provision of more oxygen; but this augmented first-stage cost is offset to some extent by consequent savings in chemical and sludge handling costs.

Considering the pH at which second-stage treatment was conducted in the pilot plant, use of an iron salt rather than alum might have been more effective. In addition, use of coagulant aids was not studied. The potential for chemical treatment warrants broader study than was practicable in this exploratory investigation.

The pilot plant as operated generated about 7 lb of waste solids per pound of BOD removed, about 10 per cent of which was excess activated sludge. Since approximately 0.6 lb of waste activated sludge solids are produced by conventional treatment per pound of BOD removed and since BOD removals through the two systems under consideration are essentially equal, it follows that relative total waste-solids production in the pilot plant was in the ratio of 7/0.6 or nearly 12 times greater. However, the volume and nature of this sludge were quite different.

Contrasted with 0.7 per cent by weight for typical conventional waste activated sludge, the solids content of mixed first- and second-stage pilot plant sludges was about 4 per cent and might be appreciably higher in a prototype installation. Its volume of 10,000 gallons per million gallons of sewage treated is actually less than the comparable

figure for conventional waste activated sludge of 12,000 gallons. However, in a conventional system the excess secondary sludge is commonly wasted to the primary clarifier where the solids are thickened and removed together with primary sludge at about 4 per cent concentration, the volume of the mixture amounting to some 5,000 gallons per million gallons of sewage treated.

The integrated process would normally be preceded by primary treatment in which about 3,000 gallons of raw sludge per million gallons of sewage treated are produced at perhaps 5 per cent solids content. Thus, in an application characterized by conditions and objectives similar to those under which the pilot plant operated, the aggregate volume of sludge produced would approximate 13,000 gallons per million gallons of sewage treated, or about two to three times that from conventional treatment. However, limited study of mixed sludges during operation of the pilot plant supports the judgment that the volume of material requiring ultimate disposal should not greatly exceed that from the traditional system. The dry weight of this material would of course be considerably higher.

Process sludge production and disposal are obviously important practical considerations which will be strongly influenced by the type of chemical treatment used in a particular application. Recognizing the important influence upon flocculation and coagulation of such interdependent variables as pH, chemical composition, alkalinity, and mixing, it must be borne in mind that results of this investigation apply to the specific substrate used. Settled Tucson sewage has a pH of about 7.5 and a total hardness and alkalinity as $CaCO_3$ of approximately 220 and 290 mg/l,

respectively. Other domestic sewages and organic wastewaters may be expected to vary greatly in their response to treatment, particularly with respect to coagulant demand. Accordingly, the quantity and character of sludge produced and methods for its disposal in a specific application of the process should receive careful study.

Sensitivity of exclusively biological treatment to shock loads is a recognized weakness. The operational flexibility of the integrated process is advantageous in this regard. With the back-up afforded by the subsequent chemical treatment system, the necessity to constantly strive for optimum activated sludge quality is eliminated and close control of first-stage treatment is not indicated. Variations in first-stage efficiency due to changes in raw-waste quality can be met quickly and without difficulty by adjustment of second-stage coagulant dosage.

Cost Evaluation

To gain some insight into the relative cost of prototype treatment plants of the type studied, cost estimates (exclusive of land) for 1-mgd and 10-mgd installations handling normal domestic sewage (225 mg/l BOD) were developed, together with figures for traditional activated sludge plants of the same capacity. Design criteria representative of good practice were used throughout. Flow diagrams assumed for each type of treatment system are presented in Figure 17.

Any such estimates are based of necessity upon various assumptions, the most important of these in this case including the following:

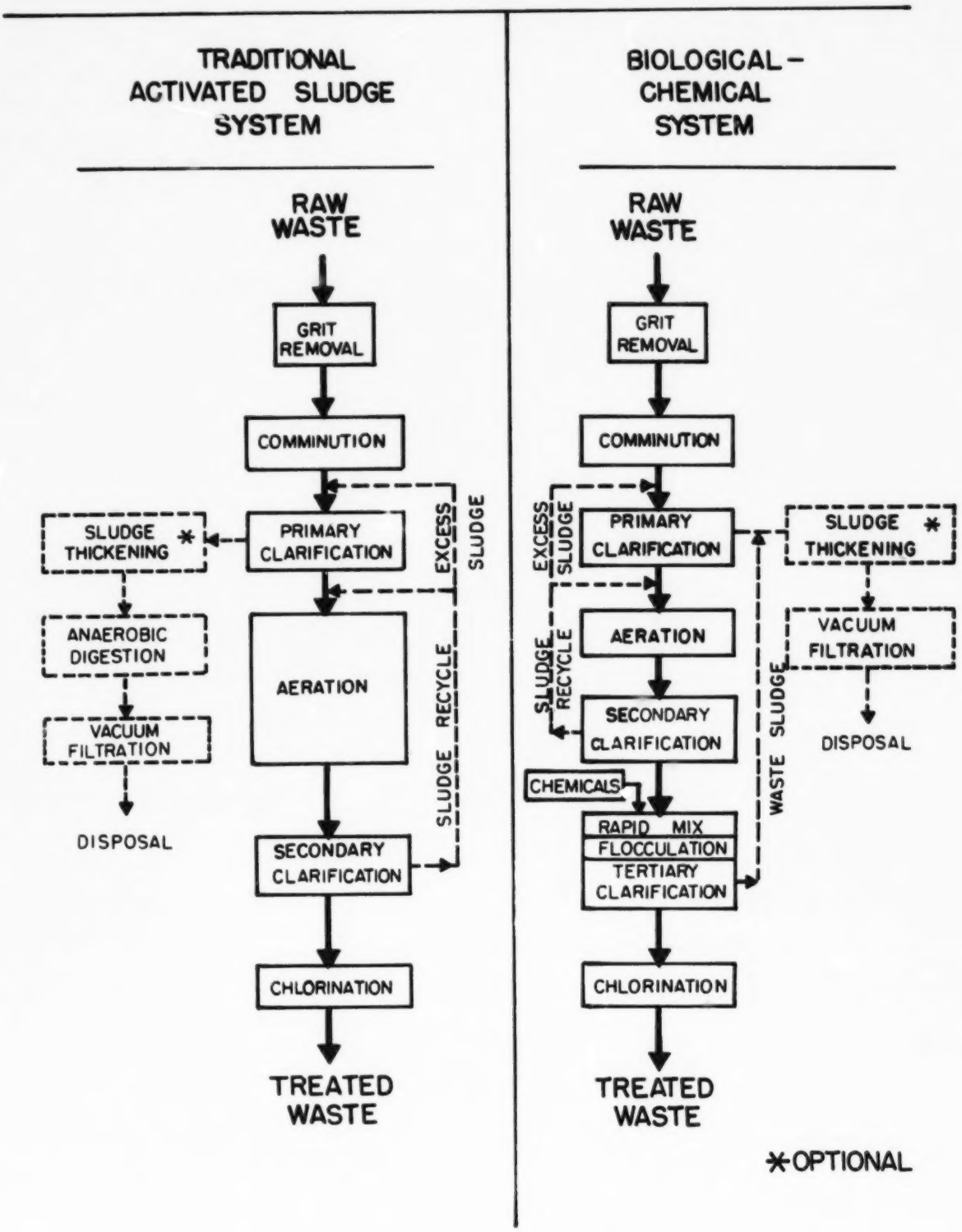


Figure 17

PROCESS FLOW DIAGRAMS ASSUMED FOR COST EVALUATION

1. Plant location: Tucson, Arizona

2. Common processes: grit removal, comminution, primary and secondary clarification, and post-chlorination.

3. Layout: a single flow line with unit plus plant by-passes for 1-mgd systems and parallel units for 10-mgd plants.

4. First-stage clarifier loadings: primary units 1,000 and secondary units 800 gpd/ft^2 at design average flow.

5. Aeration: sparged-turbine aerators for all plants. Six-hour and one-hour detention periods at design average flow for the traditional and biological-chemical systems, respectively. Respective organic loading: 0.35-0.40 and 1.5-2.0 lb BOD/lb MLSS/day.

6. Second-stage chemical treatment: flow-proportioned feed of 200 mg/l of lime (as $Ca(OH)_2$) plus 20 mg/l of alum. Volumetric feeders for alum @ $2.75/100-lb bag. Bagged hydrated lime @ $17.50/ton for 1-mgd plant. Gravimetric feeders and slakers for bulk quicklime @ $15.00/ton, including requisite chemical conveyor and storage systems for 10-mgd plant. One-minute flash mix, 20-minute flocculation, and clarifier overflow rate of 1,000 gpd/ft^2 at design average flow.

7. Post-chlorination: flow-proportioned feed of 10 mg/l of chlorine. 150-lb cylinders @ $27.00/unit for 1-mgd plants and ton containers @ $145.00/unit for 10-mgd plants.

8. Sludge handling: two-stage, heated digestion of raw sludges, followed by vacuum filtration for traditional plants. Vacuum filtration of raw, mixed waste sludges for biological-chemical plants.

9. Maintenance cost: one per cent of structures plus three per cent of equipment annually.

Cost information so developed is summarized in Table 12. Total construction costs for the traditional plants compare favorably with published information for such plants[9]. However, the indicated operating costs, exclusive of capital amortization, are somewhat higher than

TABLE 12

Cost Comparison: Traditional Activated Sludge vs. Biological-Chemical Plants

	1 mgd Capacity		10 mgd Capacity	
	Traditional Act. Sludge	Biological-Chemical	Traditional Act. Sludge	Biological-Chemical
CAPITAL COST				
Comminution and Flow Measurement	$ 6,200	$ 6,200	$ 20,600	$ 20,600
Grit Removal	8,820	8,820	27,200	27,200
Primary Clarification	17,880	17,880	109,580	109.580
Aeration, including Blowers	44,380	16,160	331,120	101,450
Secondary Clarification	19,490	19,490	122,080	122,080
Chlorination	7,350	7,350	29,630	29,630
Chemical Treatment, including Flash Mixing, Flocculation and Clarification	-	46,030	-	217,920
Piping, Valves, Pumps and Pumphouse	51,900	50,850	117,470	112,050
Administration, Lab and Blower Building	36,000	36,000	96,000	96,000
Sludge Handling:				
Thickening-Storage	-	-	21,940	21,940
Two-stage, Heated Digestion	63,840	-	219,720	-
Vacuum Filtration, Incl. Building	35,350	35,350	70,500	70,500
40% for Electrical, Legal, Administration, Engineering, Profit and Contingencies	116,480	97,650	466,340	371,580
TOTAL CAPITAL COST	$407,690	$341,780	$1,632,280	$1,300,530
OPERATING COST				
Amortization @ 4% for 20 years	$ 30,000	$ 25,140	$ 120,100	$ 95,690
Supervision and Labor	17,350	20,970	56,450	60,100
Power @ $0.01/KWH	3,900	2,800	26,500	11 050
Chemicals	6,560	13,450	36,000	82,500
Maintenance	4,370	3,430	18,250	13,190
TOTAL ANNUAL OPERATING COST	$ 62,180	$ 65,790	$ 257,300	$ 262,530

those reported[10], the difference probably being attributable in large measure to the fact that the latter reflect actual practice at operating budgets which are usually significantly less than ideal.

In any case, the ratios of costs for both types of treatment systems are considered to be reliable because similar assumptions were made throughout the comparisons.

Since the cost of land is not included, it is pertinent to note that the two-stage treatment systems studied required about 20 per cent less area than the traditional plants. At reasonable land costs, however, the influence of this advantage upon total annual cost is minor.

On the basis of the stated assumptions, the greater operating cost of the two-stage system is offset by lower construction cost and it appears that such plants can be constructed and operated at essentially the same total annual cost as that of the traditional system.

Recalling the treatment objective of nutrient (phosphate) reduction inherent in this cost evaluation, it should be noted that in many of the potential process applications not so influenced, chemical and sludge disposal costs might be reduced substantially. For example, if coagulation with say 40 mg/l of alum is practicable, annual savings in coagulant costs alone for the 1-mgd and 10-mgd plants of $3,700 and $17,500, respectively, are indicated.

The preceding example assumes production of equivalent effluent quality at low chemical dosage and it is recognized that this was not achieved in this study. Considering the lack of knowledge concerning the effectiveness of coagulation for removal of soluble materials, the merit of this assumption must await further work with wastewaters of different origin.

SUMMARY

Performance and cost evaluations of an integrated biological-chemical organic wastewater treatment system have shown its relationship to the traditional or conventional activated sludge system. It appears that the combination system is capable of producing as good or better quality effluent and that it can be constructed and operated at essentially the same total annual cost.

The system studied comprised a highly-loaded, short-detention activated sludge process followed by a flocculation-clarification chemical treatment system. Such a combination of these two processes offers important advantages in treatment stability and flexibility.

As anticipated, performance of progressively loaded bench and pilot-scale activated sludge systems treating settled domestic sewage disclosed an inverse relationship between organic load intensity and effluent quality. BOD reduction dropped from about 80 per cent to 60 per cent over a load range of from 0.4 to 4.8 mg BOD/mg MLSS/day. The percentage decrease in removal efficiency observed for other quality parameters was greater.

First-stage process stability was virtually unaffected at high loads and at the highest load studied the weight of BOD removal in a given time per unit weight of sludge solids increased nearly ten-fold over that observed at the traditional activated sludge system loading.

Severe sludge bulking was experienced at F/M ratios between 0.5 and 1.25.

The second-stage coagulation demand to achieve a given final effluent quality was a direct function of first-stage loading - the higher the F/M ratio, the greater the requisite dosage.

At high first-stage loads, appreciable dosages of the chemicals studied were needed to obtain final effluent quality comparable to that from the traditional activated sludge controls. Equivalent effluent BOD, suspended solids, and turbidity concentrations were obtained at coagulant dosages of 100 mg/l or less, but a dosage of at least 200 mg/l was needed to achieve desired final concentrations of filtered COD and TOC. Substantial additional phosphate reduction was obtained at this level of coagulant dosage.

It was observed that discharge of waste sludge from first-stage activated sludge treatment to the chemical system degraded effluent quality. However, under conditions where optimum final effluent quality is not required, the ability to so thicken waste activated sludge might be advantageous.

Solids-contact treatment in the second-stage system provided improved suspended solids, turbidity, and phosphate reductions. However, it is likely that this approach is limited to a high-pH environment in which septicity problems should not occur.

Aggregate waste-sludge solids and volume substantially greater than those from the traditional system were estimated for a prototype biological-chemical plant operated in accordance with the conditions and

objectives which characterized the pilot-plant phase of this investigation. However, the mixed sludge dewatered readily to a relatively low water content and it appeared that the final quantity of material requiring disposal would not be objectionable. Considering the influence of treatment objectives, coagulant demand, and response to chemical treatment of various organic wastewaters upon waste sludge production and disposal techniques, this aspect of each specific potential application of the combination process will require careful study.

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